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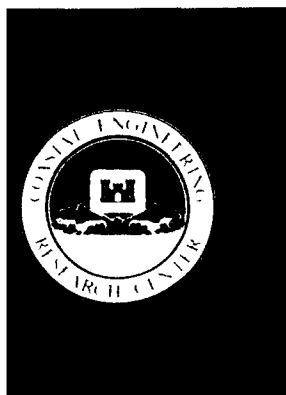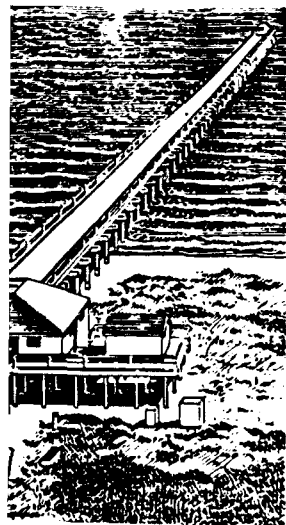
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MONITORING OF JETTY IMPROVEMENTS AT UMPQUA RIVER, OREGON
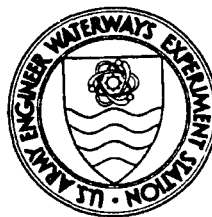
by

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13. ABSTRACT (Maximum 200 words)

Under the Monitoring Completed Coastal Projects Program, an assessment of the performance of a training jetty extension at Umpqua River, Oregon, was conducted. The monitoring was performed over the period from May 1983 through May 1984. It included the collection and analyzing of current, salinity, tide, wave, and beach and channel data. Evaluation of these data is discussed in detail. Comparison of the prototype data with that of a physical hydraulic model of the Umpqua River entrance conducted in 1970 was also performed to determine its predictive capabilities. Data on the performance of the project and observations made in the prototype, as well as conclusions and recommendations based on the monitoring effort, are reported herein.

14. SUBJECT TERMS			15. NUMBER OF PAGES 78
Beach surveys	Prototype data	Training jetty	
Channel conditions	Salinity	Umpqua River, OR	16. PRICE CODE
Currents	Tides	Waves	

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Contents

Preface

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
acres	4046.856	square meters
acre-feet	1233.6192	cubic meters
cubic feet	0.02832	cubic meters
cubic feet per second	0.02831685	cubic meters per second
cubic yards	0.7646	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
feet per second	0.3048	meters per second
inches	2.54	centimeters
miles (U.S. nautical)	1.852	kilometers
miles (U.S. statute)	1.609344	kilometers
square feet	0.09290304	square meters
square miles (U.S. statute)	2.589988	square kilometers

1 Introduction

Project Location and General Description

The Umpqua River, Oregon, Federal Navigation Project lies within the lower 12 miles[1] of the Umpqua River Estuary. The Umpqua River entrance (Figure 1) is located on the southern Oregon coast, approximately 178 miles south of the Columbia River and 404 miles north of San Francisco Bay and at approximately latitude 43°7" N and longitude 124°2" W.

The river is formed by the North and South Forks, which rise on the western slopes of the Cascade and Calapooya Mountains about 120 miles east of the Oregon coast, and it flows in a northwesterly direction to the Pacific Ocean. Surface area of the estuary is approximately 6,830 acres at mean high tide (mht) (Johnson 1972) and 20 to 30 percent of the surface area is tidelands (Marriage 1958).

Topography in the Lower Umpqua Drainage Basin is a combination of rugged, mountainous terrain to the coast and high sand dunes near the ocean. The basin terrain is quite rugged because the mountains are relatively young geologically, and are typified by narrow, sinuous valleys and steep side slopes. Relief in the lower basin varies from sea level to just under 3,000 ft. The north spit at the mouth of the river is characterized by hillocks and dunes mostly covered with dune grass and shore pine. An aerial photo of the site is shown in Figure 2. There is no vehicular access to the spit except through privately owned lands by off-road, all-terrain vehicles. The spit is undeveloped and is included in the Oregon Dunes National Recreation Area (NRA) administered by the US Forest Service. The south beach area is also in the NRA except for a small area adjacent to the river mouth. The US Coast Guard maintains a lighthouse and lookout tower on a high, stabilized dune on the south side of and overlooking the river mouth. Winchester Bay, a small, unincorporated community,

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[1]   A table of factors for converting non-SI units of measurement to SI units is presented on page v.

is located on the south side of the estuary at River Mile (RM) 1.75 (1.75 miles upstream of the mouth). Reedsport, the fourth largest city on the Oregon coast, is located at about RM 12. Primary sources of income for the residents of the area are forest products, fishing industries, and tourism. Nearly all of the area in the vicinity of the river mouth that is suitable and available has been developed for marina and associated uses, parks, and commercial and/or residential buildings.

The drainage basin of the Umpqua River system covers about 5,042 square miles and yields about 6,700,000 acre-ft of water annually with extremes of 12,000,000 and 2,750,000 acre-ft (Oregon State Water Resources Board 1958). Average river discharge is about 8,200 cu ft per second (cfs); the highest discharge of record was 265,000 cfs in December 1964.

Tides at the entrance have a diurnal inequality typical of the Pacific coast. The mean range is 5.1 ft, the diurnal range is 6.9 ft, and the extreme range is 11.0 ft. Tidal influence extends about 28 miles upstream to the town of Scottsburg. The saltwater prism is on the order of 1.6 billion cu ft; the average freshwater prism is about 313 million cu ft.

River Characteristics

Of significance to the tidal prism is the continual inflow of fresh water from upland sources. The volume of fresh water equals the discharge rate totaled over a tidal cycle; in other words, the net discharge over a tidal cycle is the freshwater discharge. Because this discharge rate varies slowly in the river during the tidal period, the ratio of freshwater volume to tidal prism is of value in the general classification of the Umpqua and other estuaries. This ratio represents the diffusion of fresh into salt water. For the Umpqua Estuary, the average ratio is 0.196, although it varies, particularly with season. This is a high value for the fresh-to-saltwater ratio, meaning that the mixing is likely to be incomplete even at the river mouth. Such an estuary, one with a high ratio, is called stratified and is a two-layered system with a distinct salinity wedge. Percy et al. (1974) classified the Umpqua Estuary as a two-layered system in January and February, when the ratio is particularly high. There are other times when the ratio is low, indicating an advanced state of diffusion. During those periods, the estuary is classified as well-mixed with only small salinity variations over its depth. It is a well-mixed system in July. Any state of partial mixing may be observed between these two extremes, and the Umpqua Estuary is classified as a partially mixed system in March, May, and October. The intrusion length, the distance from the river mouth to where the salinity is reduced nearly to zero, is a maximum, at 16.7 miles, in October during higher high water. Even at its maximum, the intrusion length is somewhat less than the 28-mile length of the estuary affected by the tides.

Navigation and Channel Improvements

Prior to improvements, the river was connected to the ocean through a gorge about 900 ft wide between the low-water lines and 1/2 mile between the high-water lines (Figure 3). On the south side of the entrance gorge, the shoreline was a rocky reef flat. On the northwest side, there was a low sand flat, bare at low tide, with generally one or two shoal passages through it. The sea entrance was obstructed by a sandbar to the southwest of the gorge about 1/2 mile from the low tide beachline. Depths in the bar channel varied from 7 to 16 ft,[1] but were seldom less than 13 ft.

The existing Umpqua River, Oregon, Federal Navigation Project (Figure 1) provides for a north jetty 8,000 ft long; a south jetty 4,200 ft long extending to a point 1,800 ft south of the outer end of the north jetty; an entrance channel 26 ft deep with no specified width; a river channel 22 ft deep and 200 ft wide from the mouth to Reedsport (approximately 12 miles upstream) with a turning basin at Reedsport 22 ft deep, 600 ft wide, and 1,000 ft long; and a side channel 22 ft deep and 200 ft wide from the main channel near RM 8 to Gardiner and a turning basin of the same depth, 500 ft wide, and 800 ft long opposite Gardiner. Also provided in the current project is a training jetty from the shoreline at the south side of the entrance to the tip of the south jetty. In addition, the project provides for access channels into a dual basin small boat moorage at Winchester Bay (RM 1.75).

Original authorization of the project was by the River and Harbor Act of 22 September 1922. It was subsequently modified by the Acts of 21 January 1927, 3 July 1930, 30 August 1935, 20 June 1938, 2 March 1945, 30 June 1948, and 3 September 1954 and by Section 107 of the River and Harbor Act of 14 July 1960 as amended. Public Law 95-482, dated 18 October 1978 (Continuing Appropriation for Fiscal Year 1979) authorized the final extension of the training jetty.

The first major effort to improve the Umpqua River entrance for navigation was the construction of a 3,390-ft-long north jetty by local interests during the period from 1916-1919. The River and Harbor Act of 21 January 1927 provided for the extension of the north jetty to its present length and for dredging of the ocean bar. Extension of the north jetty to 8,000 ft was completed in 1930 (Figure 4). This jetty was rehabilitated in 1941-1942, and a concrete cap was placed on the outer 3,977 ft. The River and Harbor Act of 3 July 1930 added a short (2,500-ft-long) south jetty, completed in 1934. Extension of the south jetty to its present length of 4,200 ft and construction of a 26-ft-deep channel were provided by the Act of 30 August 1935. These projects were completed in 1938. The Act

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[1] All elevations (el) and depths cited herein are in feet referred to mean lower low water, unless otherwise noted.

of 20 June 1938 provided for construction of the channel from the entrance to Reedsport.

As a result of a hydraulic model study, a third or training jetty was recommended and constructed in 1950-1951. This training jetty was 4,240 ft long and generally parallel to the entrance channe'. The seaward terminus of the structure was about 1/2 mile landward of the outer end of the south jetty. This construction was followed in 1963-1964 by a major rehabilitation of the south jetty (Figure 5).

In October 1978, Congress appropriated funds for construction of a training jetty extension, in accordance with the recommendations of a feasibility study conducted by the U.S. Army Engineer District, Portland (NPP). The 2,600-ft extension, which was completed in October 1980, extends to the seaward end of the south jetty and impounds about 57 acres of water. Concerns regarding water quality in and biological impacts to the impounded area led to a program to monitor water quality parameters and changes in the biological community. Since completion of the training jetty, the structures have remained in good condition and have had no detrimental environmental impact.

Previous Studies

First hydraulic model investigation - U.S. Army Engineer Waterways Experiment Station (WES)

Experience indicated that the original two-jetty system did not provide a satisfactory entrance. Specifically, ebb tidal currents met the south jetty at an abrupt angle, contributing to the deterioration and subsidence of the structure. In the search for solutions, a hydraulic model study was conducted during the period 1946-1948. The study resuited in a recommendation that a third or training jetty be constructed to solve the entrauce problems.

Second hydraulic model investigation - WES

After construction of the initial training jetty, there were shoaling problems and general dissatisfaction among project users. The presence of a channel shoal between the jetties created adverse wave conditions (Figure 6), and tug and barge operators complained of crosscurrents in the channel that tended to set the vessels onto the shoal north of the channel. As a result, a second model study was authorized in 1964. The objective of the study was to identify the optimum layout of the jetty system, include additional structures necessary, minimize the cost of maintenance dredging, improve current patterns in the entrance from the standpoint of navigation, and improve the wave climate within the entrance.

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In support of the model study, the NPP undertook an extensive data collection program in the Umpqua River Estuary in 1966 (Fisackerly 1970). The purposes of the data collection program were to (a) obtain data with which to initiate analytical studies of the shoaling problems encountered in the system, and (b) serve as a basis for adjustment and verification of the Umpqua River Estuary model. The field observations were conducted in two 32-hr periods during relatively high and low upland discharge conditions in March and August 1966, respectively. Six stations on two ranges (Figure 7) were established to collect current direction, current velocity, and salinity data. Concurrent with the current and salinity metering program, tidal elevations were measured at six locations duplicated in the model (Figure 8). Observations made during March 1966 were during a period of high freshwater discharge of 17,000 cfs. Those made in August were during a period of low freshwater discharge of 1,020 cfs. Tides during both months fell in the mean range.

Velocity measurements were obtained in the river entrance at the six stations shown on Ranges R1 and R2 in Figure 7. The remaining stations shown on that figure were used only in the model. Current speed was measured using Ice current meters and current direction was determined using remote reading magnetic compasses. Measurements were made every 45 min at the 3-ft depth, one-quarter depth, mid-depth, three-quarter depth, and 3 ft above the bottom at each station for a period of 32 hr.

The model study was conducted during 1967-1968 (Fisackerly 1970). Results of the study indicated that the best solution was to extend the training jetty to the outer end of the south jetty (Figure 9), with the crest elevation being above mean higher high water (mhhw).

Recommendations of the Committee on Tidal Hydraulics

During the early 1970's, the Committee on Tidal Hydraulics (CTH) was asked to review the project and the problems being experienced there. In its report dated November 1975 (CTH 1975), the committee concluded that even though the training jetty extension would eliminate crosscurrents, it might have caused a small increase in channel shoaling and a possible increase in wave activity in the entrance. Briefly, the recommendations of the CTH were to:

a. Rehabilitate the north jetty to reduce the amount of sediment passing through that structure.

b. Dredge the navigation channel in a more central location between the jetties, recognizing that frequent dredging would be required until the large existing shoal on the north side of the channel adjusted and stopped acting as a source of material for channel shoaling.

c. Extend the training jetty if mitigation of the crosscurrents in the entrance channel between the jetties is essential to safe vessel operation.

Monitoring Completed Coastal Projects Program

After the training jetty extension was constructed in 1980, there was a desire to evaluate its performance, and it was nominated for and accepted in the Monitoring Completed Coastal Projects (MCCP) Program in January 1983, during the program's second year. The program has as its goal the advancement of coastal engineering technology. It is designed to determine how well projects are accomplishing their purposes and resisting the attacks of the physical environment. These determinations, combined with the concepts and understandings already available, will lead to (a) upgrading the credibility of predictions of the cost-effectiveness of engineering solutions to coastal problems; (b) strengthening and improving design criteria and methodology; (c) improving construction practices; and (d) improving operation and maintenance techniques. Additionally, the monitoring program will identify concerns that laboratories should address more intently.

To develop direction for the MCCP Program, the Corps of Engineers established an ad hoc committee of coastal engineers and scientists. The committee formulated the program's objectives, developed its operational philosophy, recommended funding levels, and established criteria on procedures for project selection. A significant result of their efforts was a prioritized listing of problem areas to be addressed, essentially a listing of the program's areas of interest (Table 1). The initial list had only the first 20 items. As the program grew, three additional items were added.

The selection process has worked well since the first projects were nominated in 1981. Periodically, the Corps coastal offices are invited to nominate projects for monitoring under the program. Nominations are reviewed and prioritized by a Field Review Group comprised of representatives of Headquarters, US Army Corps of Engineers (HQUSACE), the Coastal Engineering Research Center (CERC) at the U.S. Army Engineer Waterways Experiment Station, and coastal Division offices. Final selection is based on the prioritized list of projects and available funds.

While guidance is provided by HQUSACE, management of the program rests with CERC. Operation of the program is a cooperative effort between CERC and the individual Corps District offices. Development of the monitoring plan and conduct of data collection depend on the combined resources of CERC and the Districts.

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Table 1
Monitoring Completed Coastal Projects Program Areas
of Interest

1. Shoreline and nearshore current response to coastal structures.

2. Wave transmission by overtopping.

3. Prediction of the controlling cross section at inlet navigation channels.

4. Wave attenuation by breakwaters (submerged and floating).

5. Bypassing at jettied and unjettied inlets.

6. Wave refraction and steepening by currents.

7. Beach fill project monitoring.

8. Stability of rubble structures - investigations to determine causes of failure.

9. Comparison of pre- and post-construction sediment budgets.

10. Wave and current effects on navigation.

11. Dynamics of floating structures.

12. Wave reflection.

13. Effects of construction techniques on scour and deposition near coastal structures.

14. Diffraction around prototype structures.

15. Wave runup on structures.

16. Onshore/offshore sediment movement near coastal structures.

17. Harbor oscillations.

18. Wave transmission through structures.

19. Material life cycle.

20. Ice effects on structures and beaches.

21. Model study verification.

22. Wave translation.

23. Construction methods.

2 Monitoring Program

Engineering problems in tidal waters have received continued attention from engineers and scientists in the Corps. Because of requests for Corps assistance in solving navigation problems in the Umpqua River, jetties were built and channels were dredged beyond natural depths for more economical and safer navigation. These modifications resulted in changes in flow patterns at the entrance of the estuary. These changes were analyzed and qualitative and quantitative predictions were made in model studies.

A monitoring plan was developed for the Umpqua River in early 1983 and approved in May of that year. It included the collection and analyzing of current, salinity, tide, wave, and beach and channel survey data. Current surveys were conducted in August 1983 and March 1984 in order to collect data during periods of relatively low and high freshwater flows. Beach profiles were acquired in August 1983 and April-May 1984. A self-recording wave and tide gage was deployed at about RM 0.7 in December 1983 in 22 ft of water, and this gage was operated throughout much of the monitoring period. For 1 year, while the inshore gage recorder was deployed, a Datawell bv. Waverider buoy was installed offshore of the river mouth to measure waves in relatively deep water (138 ft).

Currents

Field measurements

Freshwater flows are significant in the Umpqua River, even though their velocity is far less than tidally generated flows. The total average freshwater discharge is 313,000,000 cu ft, 19.6 percent of the saltwater volume of 1,595,000,000 cu ft. Two methods, Eulerian and Lagrangian, were used to measure the tide-generated flows superimposed on the freshwater discharges in the estuary during a tidal cycle.

The Eulerian method consisted of mechanical measurements of fluid velocities (speed and direction) past fixed points in the fluid. Velocities

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were measured using a current meter that counts the revolutions of a free-turning shaft attached to a series of cups revolving around a vertical axle mounted in simple bearings protected from the water and silt. Measurements are made by lowering the meter to the desired depth using a winch and recording the revolutions during the specified time interval. Water speed is determined from a table that relates revolutions per minute to velocity. The azimuth indicator, a separate measurement package, transmits the current direction to a remote readout on the surface. Depth was determined by a mechanical indicator attached to the reel on which the instrument cable was wound. Velocities were measured at stations A, B, and C along Range 1 (RM 0.7), stations D, E, and F along Range 2 (RM 1.9), and at a single station near the shoreward end of the training jetty (Figure 10). The location of the six stations on Ranges 1 and 2 closely approximated the stations of the prototype data acquisition program in the 1960's (Figure 7). Stations A through C were also represented in the physical model (Figure 9). Data were taken hourly at the surface, mid-depth, and bottom over a 25-hr period. Examples of the data acquired on 10-11 August 1983 at Range 1 are plotted in Figures 11-19. Since the tidal range was greater during the 1985 measurements than in the 1966 study, measured velocities during strength of flood at Range 1 were greater than those produced in the second model investigation (Figures 20-22) for the plan that recommended construction of the 2,600-ft extension of the training jetty. The tidal range during the prototype measurements in 1983 was 9 ft (Figure 23), while that used for the model was 6.6 ft (Figure 24).

In the Lagrangian method, the path followed by each fluid particle is stated as a function of time. Trajectories of water particles are tracked in space and time with the aid of tracers. The tracers (Lagrangian current indicators) used in the study were surface drogues. In August 1983, the first drouges used to describe the surface path during flood tide were free-drifting buoys with a Styrofoam core 12 in. in diameter and 13 in. thick. The buoyancy from such a large volume of foam resulted in too much surface area exposed to the wind. Fortunately, the wind velocities were small (less than 5 mph). However, it is possible that winds may have altered drogue paths somewhat. The drogues used to describe surface paths during flood and ebb tides were, therefore, modified by cutting and discarding an 8-in. thickness of Styrofoam so that they had a core 12 in. in diameter and were 5 in. thick. The modified drogues sat much lower in the water with a minimum of surface exposed to the wind.

Theoretical analysis

Freshwater flow used in the model was 10,000 cfs, over six times that in the prototype in August 1983 (1,600 cfs). This difference is significant, as will be shown first for ideal tidal flow. Waves are classified according to the ratio of the water depth, d, to wavelength, L. The dimensionless ratio d/L is called the relative depth. For linear wave theory, if the relative depth is less than 1/25, the depth is small compared to the wave

length, and the waves are termed shallow-water waves. Tidal waves have extremely long periods, so their lengths are such that the relative depth is less than 1/25, even in the deep ocean. Of particular interest in this study are the tides in the estuary where the tidal period is 12.4 hr and the maximum depth is about 65 ft.

To calculate the time-varying horizontal velocity in the x-direction for the tidal wave, an equation is derived from differentiation of velocity potential for progressive waves with respect to x after linearizing the kinematic and dynamic free-surface boundary conditions and neglecting the nonlinear terms. That equation is:

$$u = \left(\frac{gHk}{2s}\right)\left[\frac{\cosh k(d+z)}{\cosh kd}\right] \cos (kx - st) \tag{1}$$

or

$$u = \left(\frac{Hs}{2}\right)\left[\frac{\cosh k(d+z)}{\sinh kd}\right] \cos (kx - st) \tag{2}$$

where

g = acceleration of gravity

H = wave height

k = wave number

s = angular frequency

d = water depth for $z = 0$

x = horizontal distance in the direction of travel of the wave

z = vertical distance, with origin at surface

t = time

u = velocity component in the x-direction

When governed by the shallow-water condition, simplifications can be made:

$\sinh kd \sim kd$

Since n, the vertical displacement of the water surface from mean surface elevation at $z = 0$, is described by the equation

$$n = \left(\frac{H}{2}\right) \cos (ks - st)$$

Equation 2 reduces to:

$$u = sn \frac{[\cosh{(kd+z)}]}{kd}$$

however;

$$\cosh{(kd + z)} \sim 1$$

and

$$C = \frac{s}{k} \sim (gd) \, 0.5$$

so Equation 2 further reduces to:

$$u = \frac{sn}{kd} = \frac{Cn}{d} = ng^{0.5}d^{-0.5}$$

The horizontal velocity is, therefore, not a function of elevation but only of vertical displacement and water depth. Thus, the theoretical horizontal velocity for a tidal wave, using the shallow-water asymptotic forms of the hyperbolic functions and $C \sim (gh)^{0.5}$, is the same at the surface, mid-depth, and bottom. Since n is a maximum at $H/2$ or a,

$$u_{max} = ag^{0.5}d^{-0.5} \tag{3}$$

Equation 3 was derived for an ideal estuary with a rectangular cross section of constant width and depth, without consideration of energy dissipation, reflection, or resonant amplification.

For the Umpqua River entrance, a cross-sectional area of 34,800 sq ft below mean tide level was estimated. A discharge of 10,000 cfs would be associated with an average current of 0.315 fps in the absence of tidal currents. A discharge of 1,600 cfs would, similarly, be associated with an average current of 0.050 fps. During flood tide, tidal currents would be reduced by these freshwater flows. In channel sections, the maximum velocity tends typically to be on the order of 50 percent greater than the average. At the stations on Range 1, which were located in or near the channel, the maximum velocity was likely to have been more nearly 0.5 fps during a discharge of 10,000 cfs and 0.08 fps during a discharge of 1,600 cfs. For a tidal wave propagating into the Umpqua River Estuary during flood tide, the fresh water is flowing in a direction opposite the propagation of the wave. A first approximation to the waves and currents is to assume that the current is uniform over depth within the section. The maximum horizontal velocity, derived by expanding the Bernoulli equation about the free surface and making the simplifications for shallow-water waves, is:

$$u = ag^{0.5}d^{-0.5} - u'$$ (4)

where u' is the uniform current due to freshwater discharge. Equation 4 is used to compare current velocities measured in the model of the Umpqua Estuary and in the prototype when both tidal elevations and freshwater discharges were different in the model and prototype.

Comparisons of model and prototype data

A first comparison is made of currents measured during the MCCP Program and model currents measured 17 years earlier. The amplitude, a, of the tidal wave was 3.3 ft in the model and, on 10-11 August 1983, was 4.5 ft in the prototype. The average maximum velocity in the model during strength of flood was 1.3 fps at Range 1 and 3.5 fps in the prototype, as calculated from the data in Table 2. During the strength of flood (Table 2), the velocities monitored in the prototype of Station A were 1.3 to 3.3 fps greater than those predicted by the physical model. This was due to the difference in discharge because of the difference in tidal elevations. Since neither the tidal elevations nor the freshwater discharge match, it was necessary to adjust the model velocity for the effects of the differences using Equation 4 in order to properly assess the model's predictive capabilities.

Table 2
Model Currents Versus Prototype Currents During Flood Tide[1]

Station	Maximum Flood Velocity In Model (fps)			Maximum Flood Velocity In Prototype (fps) 10-11 Aug 83		
	Surface	Middepth	Bottom	Surface	Middepth	Bottom
Range 1						
A	1.2	1.8	—	4.5	4.6	4.6
B	1.1	1.8	1.7	3.2	3.6	3.0
C	0.4	1.2	0.8	3.0	3.2	2.2
Range 2						
D	—	—	—	2.8	3.1	2.3
E	—	—	—	3.4	3.2	2.1
F	—	—	—	3.9	3.2	2.5
[1] Q = 10,000 cfs in model; a = 3.3 ft. Q = 1,600 cfs in prototype during 10-11 Aug 83; a = 4.5 ft						

The first term on the right-hand side of Equation 4 ($ag^{0.5}d^{-0.5}$) is the theoretical magnitude of maximum flood velocity in an ideal estuary computed from linear theory. Unfortunately, the Umpqua Estuary is not ideal. The cross section is not rectangular in the vicinity of Range 1 and the depth is not constant. However, it appears reasonable to assume that the difference in maximum flood velocities computed for two different amplitudes would be relatively accurate since the effects of partial reflections, varying width, and varying depth for each of the computed two velocities would largely be cancelled. When the second term on the right-hand side of Equation 4 (the uniform velocity, u') is included and the difference between two maximum flood velocities computed, this difference should be accurate. Substituting values of a and u' corresponding to model test conditions into

$$u_{max} = ag^{0.5}d^{-0.5} - u'$$

gives

$$u_{max} = 3.3 \times 32.2^{0.5} 33^{-0.5} - 0.5$$

$$= 18.73 \times 33^{-0.5} - 0.5$$

$$= 3.26 - 0.5 = 2.76 \, fps$$

This theoretical calculation of u_{max} of 2.76 fps is *more than twice as large* as the average measured model velocity of 1.31 fps at Range 1. Use of theory overpredicts the velocity measured in the physical model by approximately 110 percent. This result is not surprising, since the Umpqua Estuary is definitely not ideal. The channel is of varying section from the mouth of the river upstream to Range 1.

Substituting values of a and of u' corresponding to the 10-11 August 1983 prototype data into

$$u_{max} = ag^{0.5}d^{-0.5} - u'$$

gives

$$u_{max} = 4.5 \times 32.2^{0.5} 33^{-0.5} - 0.08$$

$$= 25.54 \times 33^{-0.5} - 0.08$$

$$= 4.45 - 0.08 = 4.37 \, fps$$

This theoretical calculation of u_{max} of 4.37 fps is 23 percent greater than the average measured velocity of 3.54 fps at Range 1. Use of the theory also overpredicts the velocity measured in the prototype.

The difference in velocities predicted by the theory is 4.37 - 2.76 = 1.61 fps. When added to the measured model velocity of 1.31 fps, the estimated maximum velocity from the physical model data and theory for the August 1983 data is 2.92 fps versus a measured value of 3.54 fps. Some of this discrepancy may be due to the way that such models are calibrated. The goal during calibration is to minimize the sum of the squares of the errors (residuals) and not to match the maximum flood velocities. As was seen from model calibration graphs in Fisackerly (1970), the model under-predicted the maximum flood velocity by approximately 0.32 fps. The addition of 0.32 fps results in a predicted adjusted velocity of 3.24 fps versus a measured prototype value of 3.54 fps. Therefore, use of theory coupled with adjusted results from the physical model tests under-predicts maximum measured flood velocity at Range 1 by 9 percent.

Another way to compare the prototype measurements to model data is to compute ratios for the two conditions using Equation 4 and the actual data. This is assumed to be approximately valid since the short distance of travel of the tidal wave up the Umpqua Estuary to Range 1 means friction has only a minor effect in attenuating the wave and reducing the magnitude of current velocity.

For the theoretical calculation (Equation 4):

$$u_{model} = 2.76 \, fps$$

and

$$u_{prototype} = 4.37 \, fps$$

so

$$\frac{u_{prototype}}{u_{model}} = \frac{4.37}{2.76} = 1.58$$

For the actual data

$$u_{model} = 1.31$$

and

$$u_{prototype} = 3.54 \, fps$$

so

$$\frac{u_{prototype}}{u_{model}} = \frac{3.54}{1.31} = 2.70$$

The ratio of actual measured flow velocities (2.70) is not in good agreement with the ratio of calculated theoretical velocities (1.58). In spite of the above disparity, however, the prototype appears to be functioning as the model predicted. The model provided a sound basis for selecting a good solution.

Surface observations

Surface drogues or buoys were released and tracked through the entrance on ebb and flood tides to trace surface current patterns. Simultaneous observations were taken of the drogues at timed intervals from control stations located at each side of the estuary and drogue position determined by triangulation. During the period 10-11 August 1983, on the flood tide, the floats were released from various points across the entrance at the outer ends of the jetties. They moved in a southeasterly direction towards the training jetty, were trapped by localized eddies around the jetty stones, and beached on the jetty by the waves. A photograph of confetti floating on the water surface during the second model test (Fisack-erly 1970) was made at hr 24 during flood tide (Figure 25). During flood tide in the model, the currents moved around each of the jetties to enter the estuary. At the end of the north jetty, flow separated from the jetty. As the current moved around and headed upstream, the lack of confetti streaks adjacent to the north jetty indicated a likelihood of slack water. The tracks in the model are quite similar to those observed in the prototype. With the predominant current near the training jetty, it is not surprising that the drogues were forced onto the jetty.

During the period 21-22 March 1984 when current measurements were obtained at Ranges 1 and 2 with the Price current meter, the ebb tide drogue tracking could not be accomplished. A few days later, however, during ebb tide, the Lagrangian picture of surface currents was obtained. On the ebb tide traces, the floats were released from a point near the centerline of the navigation channel at RM 0.7. The drogues moved directly towards the inner end of the training jetty, then proceeded seaward on a track immediately adjacent to that structure. The results during ebb tide were in conformance with the model results. A photograph of confetti floating on the water surface during the second model test was made at hr 16 during ebb tide (Figure 26). During ebb tide in the model, the surface current directions proceeded directly out to sea; no eddies were apparent anywhere in the entrance. Similarly, one may have expected that the drogues in the prototype would have followed the current directly out to sea during ebb tide. In general, they moved toward the training jetty and travelled parallel to it out to sea. The movement toward the training jetty was not indicated by model tests since no transverse surface currents were evident in Figure 26. Wind or wave may have transported the drogues toward the training jetty; however, neither of these factors were simulated in the model tests for these tidal conditions.

The average surface velocity determined during the flood tide was about 3 fps for the first two releases and was greater than 4 fps for the third release. During ebb tide, the average velocity was approximately 4 fps for both releases, and the maximum velocity during any period was about 6 fps.

Other observations were obtained when a 3-year surveillance program that included monthly site visits was begun in 1977 (USAED, Portland 1982). Current data were collected alongside the north jetty during these visits. At the end of the structure, where the average water depth is approximately 20 ft, the average currents measured were on the order of 2 fps. During flood tide, the current would go around the head of the north jetty in approximately a 20-ft-wide band and into the entrance. During ebb tide, the currents moved directly seaward, as expected. Dye packets also were dropped from a helicopter in 1977 during flood tide and the dye movement observed. One packet was dropped 400 ft shoreward of the end of the jetty. In 12 min, the dye had travelled around the shore end of the jetty and into the entrance about 1,300 ft upstream of the jetty head. The total distance travelled was over 2,000 ft, indicating an average current of about 3 fps, which agrees well with the average of 3.3 fps measured using floats in August 1983. The line of dye movement was near the boundary of the larger shoal on the north side of the entrance. These surface currents were observed before the training jetty was extended to the outer end of the south jetty. Current patterns observed were noticeably similar to those seen in the second model test after extension of the training jetty. Additional information provided by the second model test (Figure 25) is the absence of confetti streaks adjacent to the north jetty, indicating possible boundary layer separation and slack water or eddy formation adjacent to the jetty.

Crosscurrents, or rotary currents, had been reported by tug and barge operators, but were not verifiable by the 1966 prototype data taken for the model study. WES personnel were able to generate currents in the model that were similar to those described by the operators and that were in the proper location in the entrance by two means. First, strong littoral current from the north was superimposed, and second, waves were generated from the northwest quadrant (Fisackerly 1970). The magnitude of the littoral current necessary to create the crosscurrents was considered to be unrealistic and the study concentrated on the wave-generated effects. The wave action appeared to build up a head differential between the "pocket" formed by the shorter training jetty and the south jetty and the navigation channel area. The crosscurrents thus generated were enhanced during flood tide as the outflow from the "pocket" turned upstream. Extension of the training jetty to the south jetty, closing off the "pocket" area, eliminated the crosscurrents. Shorter extensions reduced the magnitude and duration of the crosscurrents, but did not completely eliminate them (Fisackerly 1970). Just as there were no field data to substantiate the presence of the crosscurrents previously, there is nothing in the more recent data with which to evaluate the training jetty's efficiency in eliminating the problem. There have been no reports of continuing

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problems with the crosscurrents since completion of the extension; how-ever, tug and barge traffic has declined considerably in that time frame. Small-boat operators have always disclaimed any problems associated with crosscurrents.

Salinity

The salinity of seawater is essentially a measure of the mass of dis-solved salts in 1 kg of sea water. The mean value for the world's oceans is $S = 34.7$ g/kg, usually written as $S = 34.7$ ppt (parts per thousand). Ocean salinity is quite variable; for example, the value can approach zero near major rivers like the Mississippi or Amazon. While it is not practical to measure salinity directly, it has been demonstrated that there is a close relationship between salinity and the electrical conductivity of water, which is easily and precisely measurable. Conductivity was used to es-timate salinity in the Umpqua Estuary.

The theoretical calculations of ideal tidal flow are based on the assump-tion of small tidal amplitudes with respect to depth and a homogeneous medium without density variations due to salinity. Experimental results of tidal analysis in the WES flume tests have shown that salinity varia-tions may be neglected in tidal computations. While salinity was not a major factor, it was noticed that amplitude increased slightly with salinity. Generally, the effects of salinity may be neglected, but the estuary must be well-mixed. In a well-mixed estuary, the variations in salinity over the depth at any station must be of relative small order as compared to the mean salinity. When there are salinity variations in the estuary, the resul-tant salinity variations cause flows in addition to the tide-generated tran-sient motions, which, while primarily in the longitudinal direction, are also in the vertical direction.

Water samples were taken at each depth at each station concurrent with velocity data. The samples were later tested by the Portland District's laboratory for salinity by measuring conductivity. Two examples of the 10-11 August 1983 data are shown in Figures 27 and 28. Freshwater flow in the Umpqua River during the survey was 1,600 cfs measured about RM 56.8. Smith River and some minor streams enter the Umpqua River downstream of the gaging station, but the flow at the entrance is not sig-nificantly different from that measured upstream. The tide range during the survey was in excess of 9 ft. Under those conditions, the data indicate a well-mixed estuary with only slight variations in salinity values from the top to bottom of the water column. This indicates that flows due to den-sity variations were negligible during the period 10-11 August 1983.

The model study used a tidal range of about 6.6 ft, which approximates the diurnal range, and a freshwater flow of 10,000 cfs. An attempt was made in the model to duplicate the mixing of salt and fresh water. The comparison of the model with prototype data measured on 30-31 March 1966

is quite good (Figure 29). Compared to measurement made under this program, though, there are some inconsistencies. Stratification appears with decreased mixing through tidal action, and vertical salinity profiles become less uniform as evidenced in Figure 30 where surface salinities are markedly different from middepth and bottom salinities. Differences between middepth and bottom salinities are noticeable, although not strikingly so. The lower range, decreased mixing, and higher ratio of freshwater to saltwater prisms indicates a less advanced state of diffusion in the model, especially evident in the salinity values at the surface. In summary, the prototype data indicated only small variations of salinity over the depth. However, in the model, the surface values differed from middepth and bottom values. This was indicative of more stratification in the model than in the prototype, which would seem appropriate given the larger freshwater flow in the model.

Tides

Recording tide gages were installed at about RM 0.7 and RM 5.8 for the approximately 2 weeks during which the current measurements were made. These gages were in addition to the submerged wave/tide gage. Fisher-Porter Model 35C gages recorded the water surface elevation every 6 min. The float wells (4-in. pipes) were capped at the bottom with water entering through a multi-coil, copper tubing baffle system to eliminate, as much as possible, fluctuations due to wave action. Plots of predicted and observed entrance tidal conditions for pre- and post-construction of the training jetty are shown in Figures 31 and 32. These plots indicate that the model correctly predicted no significant changes in the tides within the estuary due to the extension of the training jetty.

Waves

A conclusion of the 1970 model study was that increased wave energy would be introduced into the lower estuary due to extension of the jetty. Some recent shoreline problems substantiate that conclusion. The wave-gaging plan called for installation of an offshore wave buoy and a gage within the estuary. Data from the offshore buoy were analyzed under contract by the University of California at San Diego, Scripps Institution of Oceanography (SIO); data from the inner gage were analyzed at CERC. Following construction of the first 4,240 ft of the training jetty in 1950-51, it became necessary to protect against wave action causing erosion just upstream of the jetty. Revetment was placed to protect the shore and served reasonably well, but in 1982 some repairs were required. The revetment is again showing some damage, primarily from wave overtopping, and further repairs are being considered. This recently verified problem lends credence to the increased wave energy premise.

Concurrent significant wave height and peak period values were obtained from the Umpqua River study for the time period 5 May 1984 to 3 March 1985. The deepwater data were measured using a Datawell Waverider buoy located in 138 ft of water about 1 nautical mile northwest of the mouth of the Umpqua River. The shallow-water data were taken with a SeaData 635-11 Wave and Tide Gage in about 22 ft of water just inside the mouth of the river.

A wave study for the Umpqua River was conducted to construct a simplistic model that could correlate wave conditions in the estuary with those outside the estuary system. A second objective of the wave study was to obtain results that hopefully could be extrapolated to similar estuary systems.

In the analysis of the Umpqua River entrance, an empirical formula with a single coefficient that accounts for the net energy reduction for deep-water waves entering the mouth of the river was used to estimate the shallow-water wave height.

The total average energy per unit area (equal to the magnitude of the sum of the potential and kinetic energies) is given by

$$E = \frac{wH^2}{8} \tag{5}$$

which is transmitted across the unit area with the group velocity C_G, and where w is the specific weight of water and H is the shallow-water wave height.

The average energy flux in the direction of wave propagation is

$$P = EC_G \tag{6}$$

The group velocity changes from deep water into shallow water according to

$$C_G = nC \tag{7}$$

where C is the wave celerity, and n is the transmission coefficient which, according to linear wave theory, is given by

$$n = 0.5\left(\frac{1 + 2kd}{\sinh 2kd}\right) \tag{8}$$

where k is the wave number and d is water depth. The factor n thus has as deep- and shallow-water asymptotes the values of 1/2 and 1. In deep water, then, the energy is transmitted at only half the celerity of the wave, whereas in shallow water the energy and wave form travel at the same speed.

It follows that the power of transmission is

$$Pb = EbC_G = \frac{wH^2b\,(nC)}{8} \tag{9}$$

where b = width between orthogonals in shallow water.

In deep water, just before the waves are due to feel the effects of depth, the above equation becomes

$$P_o b_o = \frac{wgH_o^2 b_o n_o C_o}{8} \tag{10}$$

with the subscript o always referring to the deep-water condition.

Taking the ratio of the power of transmission in shallow and deep water (Equations 9 and 10) results in

$$\frac{H}{H_o} = \left(\frac{b_o}{b}\right)^{0.5}\left(\frac{n_o C_o}{nC}\right)^{0.5}\left(\frac{Pb}{P_o b_o}\right)^{0.5} \tag{11}$$

which may be expressed as

$$\frac{H}{H_o} = K_r K_s K_{fp}$$

where K_r, K_s, and K_{fp} are known as the refraction, shoaling, and friction-percolation coefficients, respectively.

Thus the shoaling coefficient, according to linear wave theory, is given by:

$$K_s = \left(\frac{n_o C_o}{nC}\right)^{0.5} \tag{12}$$

In English units, this expands to

$$K_s = \frac{0.903\,Tk\;0.5}{2n\;0.5}$$

$$\tag{13}$$

$$= \frac{0.903\,Tk^{0.5}}{\left(\dfrac{1+2kd}{\sinh 2kd}\right)^{0.5}}$$

where T is wave period. Therefore,

$$H = \left[\frac{0.903\, H_o T k^{0.5}}{\left(\dfrac{1+2kd}{\sinh 2kd}\right)^{0.5}}\right]\left(\frac{b_o}{b}\right)^{0.5} K_{fp} \tag{14}$$

If one includes a coefficient K_* to account for the sum of reflection and diffraction, Equation 14 becomes

$$H = \left[\frac{0.903\, H_o T k^{0.5}}{\left(\dfrac{1+2kd}{\sinh 2kd}\right)^{0.5}}\right]\left(\frac{b_o}{b}\right)^{0.5} K_{fp} K_* \tag{15}$$

The expanded shoaling coefficient contained in the right side of the above equation is a function of wave length, water depth, and period. Standard textbooks on wave mechanics state that, as waves move into shallow water, their period T remains constant. However, severe nonlinear deformation can affect the apparent wave period by causing the incoming wave crest to split into two or more crests. This effect is common in laboratory experiments and is also expected to be ·mon in the prototype. Also, the peak period generally change because of a shift of energy distribution within the sea state a waves propagate into shallow water from deep water. Hence, the peak period merits inclusion as one of the independent variables. If the depth were held constant, it seems plausible to assume that the shallow-water wave height would be a function of the product of the deep-water wave height, period, and square root of wave number. Thus a polynomial approximation based on the variable $H_o T k^{0.5}$ might be used to estimate the shallow-water wave height from the deep-water wave height. The empirical relationship of the deep-water wave height H_0 measured by the Datawell Waverider buoy and the shallow-water wave height H measured by the SeaData 635-11 Wave and Tide Gage is of the form

$$H = CH_o T k^{0.5} \tag{16}$$

If X is set equal to $H_0 T k^{0.5}$, then the model used in this study becomes

$$H = CX \tag{17}$$

where C is an empirical constant to be estimated.

The prototype data were processed in two different ways. Daily average significant wave heights and peak periods were computed to reduce the two data sets to matched pairs relative to time of measurement. Daily maximum significant wave heights may be of more general interest for planning purposes than daily averages. For this reason a correlation was also computed for pairs of daily maximum significant heights and associated peak periods.

For both the daily averages and the daily maximum shallow-water significant height displays, there was an approximately linear trend for smaller values of the variable X, as seen in Figures 33 and 34. For larger values of X, the shallow-water significant height appears to be constant. One explanation for this apparent lack of correlation in the larger values of X is that the larger waves do not enter the mouth of the river. Another possible explanation is that larger waves entering the harbor mouth undergo a more radical loss of energy than do the smaller waves, but without wave direction information, it is not possible to further explore this aspect of the results.

The model developed is a site-specific model for the Umpqua River Estuary. The straight line regression for variable H versus variable $X = H_0 Tk^{0.5}$ is defined by a line that gives the best estimate of H for a given value of X. The regression line was fitted by the method of least squares of the departures from the line. The regression output for daily averages is

$$H = 0.082 H_o Tk^{0.5}$$

The confidence interval is defined as an interval around the computed parameters within which a given percentage of parameters of a large number of samples is expected to be found. This given percentage is the level of confidence. The confidence interval at the 90-percent level means that, out of 100 samples of equal size, it is expected that 90 values of a parameter would be inside that interval. The confidence limits are numerical values describing the boundaries of the confidence interval. The 90-percent confidence limits on $C = 0.082$ are $(0.079, 0.084)$.

One measure of linear correlative association is the correlation coefficient, r,

$$r = \frac{(\Sigma X_i H_i - N\overline{X}\overline{H})}{s_x s_H (N - 1)}$$

where s_x and s_H are the standard deviations of X_i and H_i, respectively, and the quantities H and X represent the means for the variables $\overline{H}$ and $\overline{X}$. The correlation coefficient is the most commonly used statistical parameter for measuring the degree of association of the two linearly dependent variables H and X. The correlation coefficient is unity only if all points fall on a straight line. A positive value of r means that H increases with an increase of X. If there is no linear relationship, $r = 0$. If there is a functional linear relationship, $r = +1$. All values of r between these limits describe the various degrees of correlative association. The greater the absolute value of r, the greater the linear correlation. For daily averages, the correlation coefficient was $r = 0.68$.

The regression output for daily maxima is

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$$H = \ldots Tk^{0.5}$$

The 90-percent confidence interval on $A = 0.11$ is (0.102,0.116). The correlation coefficient is $r = 0.35$.

The coefficient of correlation r takes on a value of 0.68 for daily average significant height. This implies that 68 percent of the variation in shallow-water daily average significant wave height can be explained by the variation in $H_o Tk^{0.5}$. The daily maxima resulted in a much lower r, implying that only 35 percent of the shallow-water variation can be explained by deep-water variation.

The significant wave height $H_{1/3}$, is defined as the mean of the highest one third of the waves present in the sea. This is ostensibly the magnitude of the average wave height that a conscientious experienced observer will estimate. Experimental results show that when the wave shapes are not severely deformed by shallow depth or high wave steepness, the significant wave height is approximately equal to the zero moment wave height $H_{mo} \sim 4s$, where s is the standard deviation of the wave record. It can also be shown that the variance of the wave record s^2 is equal to one half the sum of the squares of the Fourier coefficients

$$s^2 = 0.5 \Sigma a_i^2$$

where a_i are amplitudes; i.e., the Fourier coefficients. The premise of spectral analysis is that the waves recorded at a wave staff are composed of components of many frequencies and amplitudes with different phases. Each of the components are simple harmonic in time and are mutually independent. Thus measured wave data can be analyzed to determine the unknown phase and amplitude characteristics of each component for the wave record.

The procedure of extracting spectra from wave records for this study was performed, and the significant height was calculated using the above formula. The wave period corresponding to the highest peak of the spectrum was also extracted. Peak period has greater dynamic importance than significant period, although the two parameters are generally comparable.

It should be noted that the crests of significant waves do not maintain their identity and are not conservative in the storm area. Using the equations based on classic linear (small amplitude) wave theory, the designer would likely select a single average height and period (significant height and period) and propagate it into the Umpqua Estuary as if it were a linear wave. (The designer has, in effect, statistically characterized the sea simplistically through his use of significant waves). Significant waves are "statistical disturbances" and, as such, have no crests whose identity can be maintained. It is possible that they may not propagate like waves at all, and it is more unlikely that they propagate into Umpqua Estuary

similar to monochromatic waves. (A designer should keep this in mind when propagating a deep-water wave into shallow water and computing shallow-water wave heights to the nearest one-hundredth of a foot utilizing linear wave theory). Inaccuracies are inherent in computation of shallow water significant wave heights using equations based on small-amplitude wave theory.

Wave models, similar to the model developed for the Umpqua River Estuary, could be applied to other estuaries and harbors along the Pacific coast with a variety of differing characteristics. If enough regression models were developed, a regional model for predicting shallow-water wave heights in an ungaged estuary or harbor might be possible using the multiple regression method and physical estuary or harbor characteristics such as entrance area, tidal prism, and mean estuary or harbor depth.

Beach Surveys

Beach profiles were taken north and south of the entrance at 15 stations over a length of about 1 mile to the north, and at 8 stations over a length of 1/2 mile to the south. The stations were originally established as part of an evaluation of the earlier north jetty rehabilitation. Therefore, greater emphasis was placed on the north beach. This study incorporated the earlier data and duplicated the locations to facilitate comparisons in August 1983 and May 1984. Surveys over the dry beach were made with conventional land-surveying techniques. Soundings through the surf zone were done with a helicopter. A 3/8-in. cable, with colored balls and cubes marking 1- and 10-ft intervals and a lead weight at the end, served as the "rod." The cable was suspended on a pulley assembly below the helicopter and lowered until the weight touched the bottom, activating a signal in the helicopter cockpit. The helicopter then hovered momentarily while survey personnel on shore read the rod. Distance was determined with an AGA 140 electronic distance-measuring instrument aimed at a cluster of prisms mounted on the aircraft. The instrument reduces the slope distance to a horizontal distance. The helicopter started offshore and proceeded shoreward along a marked range line with soundings taken at about 75- to 100-ft intervals. The profiles extended from approximately the top of the dune to beyond the -30-ft contour. The bulk of the seasonal onshore-offshore sediment movement is estimated to occur within that zone. Surveys were made in the spring of the year and again in the fall.

The original intent of the surveys was to determine if the north jetty rehabilitation was effective in preventing movement of material through the structure. There is little doubt that the rehabilitation reduced the permeability of the jetty and had some impact on the amount of material moving into the entrance. However, the beach survey data does not, in itself, provide documentary proof. The first survey in the series was made in September 1977, the time the north jetty rehabilitation was being completed. A comparison of the data of that survey to that of August 1979,

which was prior to any significant extension of the training jetty, shows an overall loss of material of about 3 percent on the north beach. That is not considered to be within the accuracy of the survey method; however, the data do indicate a slight increase in volume in the first 1,400 ft north of the jetty, which might indicate some reduction in the amount of material passing through the structure. A comparison of data from the fall survey of 1977 to that of 1983 shows much the same trend (i.e., an overall loss of material but a slight increase in the immediate vicinity of the jetty). Between the fall of 1977 and the spring of 1984, the south beach showed a gain in total volume of about 29 percent, whereas the north beach showed a loss of approximately 7 percent in that same time frame. However, most of the volume figures computed from the survey data tended to cluster around a mean value on both beaches. If the premise that the significant movement of sediments is confined within the limits of the surveyed area is true, then the total volume of material within those boundaries might be expected to remain fairly constant. While the methodology of obtaining the survey data in the surf zone is considered reasonably good under difficult conditions, there are some questions concerning the ability to repeat a survey. Based on the data measured by these techniques, the conclusion is that none of the structural improvements at the entrance affected the adjoining beaches in any significant manner.

The beach material is a fine-grained, uniform sand with an average (d_{50}) size of about 0.26 mm (0.0102 in.). The d_{50} size on the north beach ranged from 0.22 to 0.32 mm and from 0.21 to 0.36 mm on the south beach. The material on the south beach, adjacent to the entrance, was slightly coarser than that farther down the coast; the north beach material was fairly uniform spatially.

Channel Conditions

Hydrographic surveys are taken routinely as part of navigation project maintenance. Surveys of the navigation channel are taken in the spring and again in the fall at a minimum, and normal practice is to obtain additional surveys periodically through the summer season, especially for the more highly utilized projects. Sea and weather conditions make it impractical to conduct surveys during the winter months. Routinely, the surveys consist of 10 to 12 lines at 50-ft spacing paralleling the channel centerline. The coverage extends one line or more outside the channel prism lines. Surveys are made periodically that essentially cover a "bank-to-bank" area from about RM -0.2 to the seaward end of the jetties. Surveys are made by floating craft using echo sounding techniques. Horizontal control is by electronic positioning devices and vertical control is by a tide gage located at about RM 0.7.

This section differentiates between the entrance and the Federally maintained navigation channel. The term channel as used herein refers to the

navigation channel, whereas the term entrance refers to the full hydraulic width between the jetties.

Dredging Records

Maintenance dredging records for the periods of concern were also examined in the evaluation. There are considerable variations in quantities from year to year and there are hazards involved in placing too much emphasis on dredging records. Many times, factors other than the amount of shoaling may influence the dredging quantities. Factors such as availability of equipment, funding limitations, environmental conditions, commitment of resources to other projects, variations in the amount of advance maintenance accomplished in the previous dredging, etc., may affect the maintenance dredging figures. The 1980 eruption of Mt. St. Helens, resulting in the commitment of all available equipment to the Columbia River project, is an extreme case in point. Table 3 shows the volume of material removed from the navigation channel at the entrance of the Umpqua River. The years 1975 through 1977, 1978 through 1980, and 1981 through 1983 were selected as representative of the three periods of concern. The extremely low figure for 1977 was due to adverse weather conditions and tends to bias the average for that period.

Table 3 Umpqua River Maintenance Dredging Quantities			
Calendar Year	Quantity, cu yd	Total	Average
1975	242,640		
1976	201,660		
1977	75,300	519,600	173,200
1978	189,500		
1979	317,532		
1980	294,450	711,482	237,161
1981	228,091		
1982	246,190		
1983	135,950	610,231	203,410

Bathymetric Changes

Computations of volumetric changes following structural modifications or major rehabilitations were made using an arbitrarily selected datum plane. The plane was selected at a depth sufficient to ensure detecting any change and well below any dredging or other non-natural activities. The channel computations were made from the initial spring survey for each year, but as the full-width entrance surveys were made periodically, all such surveys between 1977 and 1984 were used.

The channel reach examined was about 3,500 ft in length extending from about RM -0.3 to a point about 300 ft seaward of the south jetty terminus. No discernible changes in the channel were noted upstream of that reach. Comparisons were made between the pre-north jetty rehabilitation conditions, the pre-training jetty extension conditions, and post-training jetty extension conditions. There was a short time period between the rehabilitation of the north jetty and the extension of the training jetty; therefore, the data for the first period are limited. For each time period, volumetric computations from 2 years of bathymetric data were averaged. Comparison of the results shows a reduction of about 8 percent between the pre- and post-north jetty rehabilition and about a 7-percent reduction between the before and after training jetty extension periods, after correcting for material removed by maintenance dredging. At a point about 700 ft upstream of the south jetty terminus, the survey data imply a greater influence on channel development from the training jetty extension. For that specific location in the channel, that conclusion is likely to be correct. However, for the full reach examined, the data indicate a nearly equal reduction in material following each improvement.

For the entrance, surveys covering a reach about 1.1 mile in length from RM 0 seaward, and essentially the full width of the entrance, were used for computations. For those surveys predating the training jetty extension, only the area north of the extension alignment was considered, so as to be consistent with the post-extension computations. The surveys examined, totaling 18, covered the period from July 1977 to July 1983. The data show a significant reduction, about 18 percent, following the north jetty rehabilitation. In the period since extension of the training jetty, the data show only a slight reduction, on the order of about 4 percent. Volumetric changes over the entire entrance (full width) suggest the north jetty rehabilitation was the primary cause of reduction of material while the training jetty extension was of relatively minor importance. However, if one confines the scope of the computations to only the navigation channel, then the data show a nearly equal reduction of material following each improvement.

With the present entrance configuration, the narrow point of the entrance, about 1,450 ft, is located about 1 mile upstream of the outer end of the south jetty. The cross-sectional area of the entrance is about 34,800 sq ft at mean tide level (3.7 ft above mean lower low water

(mllw)). The average depth is approximately 22 ft below mllw; and in the reach between the tips of the jetties and RM 1, depths range from 5 to over 70 ft below mllw. Depths of 45 ft and greater have developed adjacent to the extended training jetty, and near the outer end of the south jetty, depths are in excess of 50 ft. Small shoals with depths 2 to 3 ft less than the 26-ft project depth encroach from the north but do not extend into the channel to any great extent. The project authorized no specific width of channel, and, prior to the extension of the training jetty, the District attempted to maintain the channel at 200 ft, consistent with the upriver channel. Presently, the channel is maintained at 300 to 500 ft in width.

The extension of the training jetty was completed in the fall of 1980. The first survey the following spring showed a controlling depth along the north channel line of 19 ft. In an effort to lessen the effects of the northside shoal, the channel was rotated approximately 19 deg to the south. The spring surveys of 1982 and 1983 show the controlling depth to be 23 to 24 ft. In May 1984 a shoal with a controlling depth of 21 ft appeared. That depth existed on the channel's north prism line and only two or three soundings reflected that depth. For the most part, the depths are on the order of 23 to 25 ft. While the 1984 survey revealed depths less than the project depth of 26 ft, the post-1980 surveys showed a much-improved channel compared to the pre-1977 conditions. That is not to say the project does not have problems, as the bar can still be extremely rough and hazardous to navigate under some conditions. The large shoal lying between the channel and the north jetty continues to act as a source of sediments to the channel shoal, and waves steepening and breaking as they pass over the shoal create problems for small boats. The shoal is about 1 mile in length and approximately 1,000 ft wide at its widest point. It begins a little seaward of the outer end of the north jetty and extends into the entrance along the north side. Depths vary from 1 ft or less to 20 ft at zero tide. It would require extensive dredging to significantly reduce the shoal, and resources in the past have been too limited to try. It is highly unlikely that those circumstances will change in the foreseeable future.

The model study indicated a potential for increased channel shoaling, primarily in the reach adjacent to the extended portion of the training jetty. That increase has not yet occurred. The model study was conducted on a fixed-bed, distorted-scale model, and it is recognized that such models have limited capabilities for evaluating shoaling. The channel has developed as anticipated with a marked improvement in conditions. There is no documentation of users' views, but personal communications indicate their reaction is generally favorable, although not unanimous. Some small boat operators were opposed to the training jetty extension because it was their practice to use the area closed off by the structure as a holding area where they could observe bar conditions and possibly cross during a lull in the wave action. In the relatively short time since structural changes at the entrance, the outer bar has demonstrated nothing in the way of changes that could be called a trend. In the spring of 1981 and 1982 the outer bar was essentially nonexistent, but in 1983 and 1984 it was again

present and appeared little different from pre-1977 surveys. Its location varies between 1,000 and 1,500 ft seaward of the tips of the jetties and normally extends across the channel, or nearly so. Depths range from 20 to 25 ft below mllw. The data set shows the bar may tend to form at the more landward location more often than previously, but that is based on limited observations. There were no significant impacts on the outer bar expected for either the north jetty rehabilitation or the training jetty extension, and to date that assumption has proven to be correct.

Inlet Stability

The tidal prism - inlet area relationship has been documented by several authors over the years (most notably M. P. O'Brien, 1931) and while the relationship does not take into account some seemingly important parameters, it is a fairly reliable indicator. The tidal prism for the Umpqua Estuary is 1.595×10^9 cu ft (Johnson 1972). Other researchers have expanded on O'Brien's work, and the tidal prism - inlet area relationship for a two-jetty system on the Pacific coast with diurnal tides is given by $A = 5.28 \times 10^{-4} P^{0.85}$ where A = the entrance area in square feet and P = the tidal prism in cubic feet (Jarrett 1976). From that relationship using the reported value of tidal prisms, the inlet area computes to be 35,000 sq ft. Such close agreement between the empirically developed relationship and the actual area must be fortuitous to some degree but it does suggest the validity of the relationship and that the Umpqua entrance is approaching a state of dynamic equilibrium under the newly imposed conditions.

Applying some empirical relationships for stable channels in alluvial material yields interesting, albeit inconclusive, results. Those relationships, sometimes referred to as "regime theory," were developed primarily from studies of irrigation canals in India and Egypt. The equations relate channel geometry to discharge and, in some direct or indirect manner, to sediment characteristics. Predictably, most of the relationships developed from the early studies were based on limited data from relatively similar channels and thus contain some inherent weaknesses.

Tidal discharge Q may be approximated by:

$$Q = A_c V = \frac{2 A_b h}{T} \quad \text{(Sverdrup, Johnson, and Felming 1942)}$$

where

A_c = entrance cross-section area

V = average velocity

A_b = surface area of bay

h = tidal range

T = tidal period (=12.42 hr for semidiurnal tides of Pacific coast)

$A_b h$ = the product of the bay surface area and the tidal range is defined as the tidal prism, P.

Assuming the tide is sinusoidal, $Qmax$ would be $\pi/2$ times $Qavg$ or $A_b h\pi/T$. For a tidal prism of 1.595×10^9 cu ft, $Qavg$ and $Qmax$ would be approximately 71,400 cfs and 112,000 cfs, respectively. Bruun and Gerritsen (1960) in their analysis of inlet stability concluded that it is the maximum discharge which controls channel stability. The average annual freshwater discharge of Umpqua River is about 1/10 the average tidal discharge and is ignored in the following computations, although winter flows commonly reach 30,000 cfs or more and may be a factor in the channel geometry.

Lacey (1929) developed the following equation relating discharge and velocity:

$$Qf^2 = 3.8 \, V_{cr}^6$$

where

f = Lacey's "silt factor" = $8(d)^{0.5}$

V_{cr} = critical velocity to prevent shoaling

Q = discharge

d = median diameter of bed material, in inches

Replacing the velocity term by Q/A and assuming d equals 0.0118 in., an expression $A = 1.31Q^{0.83}$ can be developed. (In the balance of this discourse, unless otherwise noted, A shall mean the area of the entrance). For a discharge of 112,000 cfs, Lacey's equation gives an area of 20,300 sq ft, about 60 percent of the actual area of the entrance and that predicted by Jarrett's tidal prism-entrance area relationship. It would require an additional 100,000 cfs discharge for Lacey's equation to result in an area of 35,000 sq ft.

Blench (1961) developed the following equations which include what he termed "bed factor" and "side factor" (F_b and F_s). Sediment characteristics are taken into account by these factors.

$$\frac{V^2}{D} = \frac{F_b \text{ and } V^3}{W} = F_s$$

where

D = depth of flow

W = mean width of entrance (Such that $WD = A$)

V = velocity

The above expressions may be rewritten as:

$$W = \left(\frac{F_b}{F_s}\right)^{0.5} Q^{0.5}$$

and

$$D = \left(\frac{F_s}{F_b^2}\right)^{0.33} Q^{0.3}$$

and

$$WD = A = Q^{0.83}(F_b F_s)^{1.67}$$

F_s is assumed to be 0.1 for noncohesive sand (Mason 1972) and F_b is $1.9(d_{50})^{0.5}$ $(1 + 0.012c)$ where c is bedload concentration in parts per million and d_{50} is in millimeters. For $d_{50} = 0.3$ mm and no bed load, Blench's equation becomes $A = 1.46Q^{0.83}$ which is close to Lacey's equation. For a discharge of 112,000 cfs, Blench's equations give a value of about 22,700 sq ft for the area. The assumption of zero bed load may not be valid; however, any positive value for c would result in a larger discrepancy between the calculated area and the actual area or that predicted by Jarret's equation. Blench's equations for the width and the depth give values of approximately 1,100 ft and 22 ft, respectively. The depth figure agrees well with the average depth (flow area divided by the width) in the Umpqua entrance but that may well be coincidental, given the disparity of the calculated width and area values with actual entrance geometry. It should be noted that the jetties at the entrance form what are essentially rigid and unnatural boundaries.

Simon and Albertson (1963) developed a relationship for a stable channel area from their analysis of data from a larger and more varied number of sources (Henderson 1966). Simons and Albertson developed two relationships for area depending on the size of the hydraulic radius. For an inlet with a radius greater than 7 ft, the relationship is

$$A = 1.8\ K_1 K_2 Q^{0.5} + 0.84\ K_1 K_2 Q^{0.86}$$

For sand, $K_1 = 3.5$ and $K_2 = 0.52$ (Mason 1972) and the equation becomes

$$A = 6.3\ Q^{0.5} + 1.53\ Q^{0.86}$$

For Umpqua River, the above equation results in an area of 35,800 sq ft which agrees very well with the measured area and with that predicted by the Jarrett equation. The Simon and Albertson equation appears to yield somewhat better results than either the Lacey or the Blench equations based on this extremely limited evaluation. However, the greater number and more varied types of channels examined by Simon and Albertson should tend to increase the reliability of their findings.

Table 4 shows the predicted channel area by both the Jarrett and the Simon and Albertson equations for other entrances on the Pacific coast using Johnson's reported values for tidal prisms. The table shows that the Jarrett and Simon and Albertson equations agree very well for all the listed entrances. It also shows that those predicted values vary from Johnson's measured areas by about 7 to 32 percent and are about equally divided between over- and under-predicting Johnson's values; Umpqua River shows the closest agreement among the three values. Some of the disparity may be accounted for in the scale of the charts used to measure the area and the amount of bathymetric data available on those charts. Examination of more detailed hydrographic survey data and computation of channel area for the various entrances is needed to further evaluate the Simon and Albertson equation. Unfortunately such detailed surveys of the entrances are rarely if ever obtained. It is encouraging to note the remarkably close agreement of the Simon and Albertson equation with that of Jarrett's, given the degree of validation the latter relationship has received.

Table 4
Comparison of Predicted Channel Area

Entrance	Tidal Prism[1] cf	Area[1] A_{jo} sf	Area[2] A_j sf	Area[3] A_{sa} sf	Ratio A_j/A_{sa}
Coquille River, Oregon	1.77×10^8	7,030	5,400	5,800	0.93
Siuslaw River, Oregon	3.66×10^8	8,330	10,000	10,500	0.95
Yaquina Bay, Oregon	1.12×10^9	19,610	25,900	26,500	0.98
Umpqua River, Oregon	1.60×10^9	33,000	35,000	35,800	0.98
Coos Bay, Oregon	2.15×10^9	56,500	51,700	52,400	0.99
Humboldt Bay, California	3.51×10^9	51,900	68,600	69,500	0.99

[1] Johnson (1972).
[2] Jarrett (1976).
[3] Simon and Albertson (1963).

3 Water Quality and Biological Communities

Though not a part of the MCCP Program study, a brief description of the evaluation of possible water quality and biological changes in the impounded water area is valuable. Extension of the training jetty to the south jetty head impounded about 57 acres of nearshore ocean environment. During preparation of plans for the extension, concerns were raised about potential impacts to water quality, benthic organisms, and fish in the impounded area. Four culverts were placed through the jetty at about the mean tide line, and a section of the training jetty about 200 ft in length was purposely constructed with increased permeability to enhance flow to provide for fish passage. A monitoring study was initiated in 1980 prior to closing off the area and was completed in 1983. Water quality, benthic organisms, and fish samples were collected. Elements of the study were conducted by NPP personnel and by the Marine Biology Department of the University of Oregon. The data suggest the impacts of the impoundment were as follows:

a. Dissolved oxygen content, pH, and oxygen reduction potential remained within acceptable standards, and there were no discernible changes in those parameters associated with the jetty extension.

b. The impoundment area was well-mixed, and salinity values were correlated with the marine environment.

c. A minor increase in surface temperature was noted during high tide. Throughout most of the tidal cycle, temperature profiles closely resembled those of the reference stations.

d. The benthic invertebrate community was altered by an increase in the abundance of capitellid and spionid polychaetes, bivalves, and crustaceans, and a decrease in the abundance of dungeness crab and bay shrimp.

e. An increase in the abundance of english sole and a decrease in several species of midwater fish was noted.

The data indicate that no problem in terms of water quality occurred due to the jetty extension, and that while there were changes in the biological communities, there were no irreversible damages.

4 Conclusions and Recommendations

Conclusions

Anticipated results from the jetty extension have been realized to a large degree, and there have been no significant deleterious effects. The physical hydraulic model appears to have done a good job of predicting post-construction conditions, with the possible exception of shoaling, which is the weakest element of a fixed-bed model.

Based on the analysis of the data collected during this study and others, the following specific conclusions were reached:

a. The channel has improved in terms of depth and, to a somewhat lesser degree, width, as anticipated.

b. There were no significant deleterious impacts to adjacent shorelines. tidal or salinity regimes, or current patterns.

c. The physical hydraulic model proved to be an excellent predictive tool for hydrodynamics and salinity changes. Increased channel shoaling predicted by the qualitative shoaling studies has not manifested itself.

d. Regime theory or appropriate inlet stability analysis is important in tidal inlets on sandy coasts where maintenance dredging may be needed.

Recommendations

The success of the improvements at the Umpqua River entrance is a testimony to the tools used. Recommendations resulting from the study are:

a. Districts should use physical models to test alternative designs for coastal river entrances. The models have proved to be extremely reliable in predicting phenomena occurring at coastal sites.

b. Consideration should be given to making jetties at tidal entrances generally parallel to each other and the navigation channel. Converging, or "arrowhead," jetties often fail to provide for stable entrances and safe navigation.

c. Further work on inlet stability analysis should be undertaken, to confirm the applicability of "regime theory" to tidal entrances.

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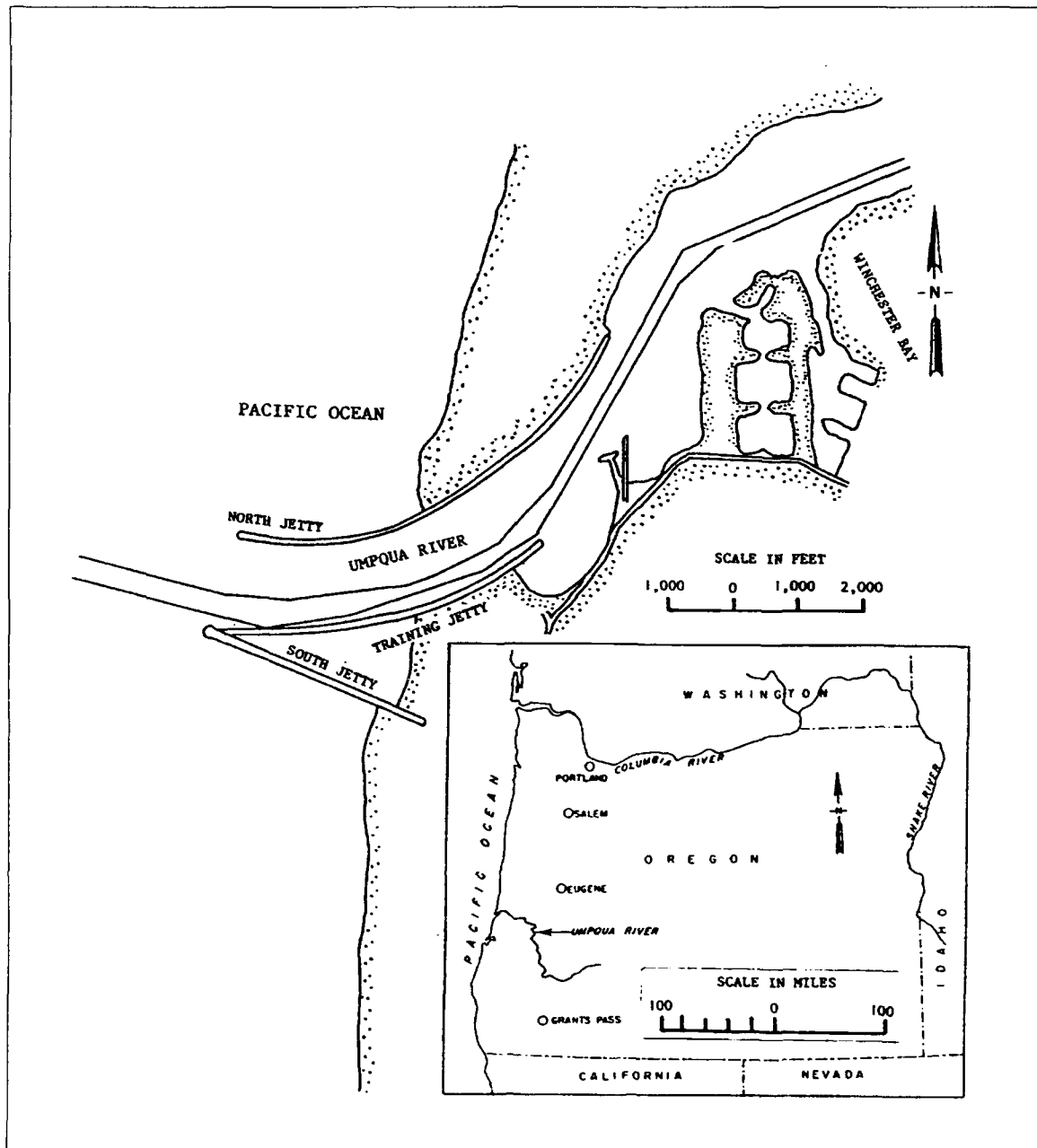
Figure 1. Project location and existing river entrance

Figure 2. Aerial view of Umpqua River entrance (September 1983)
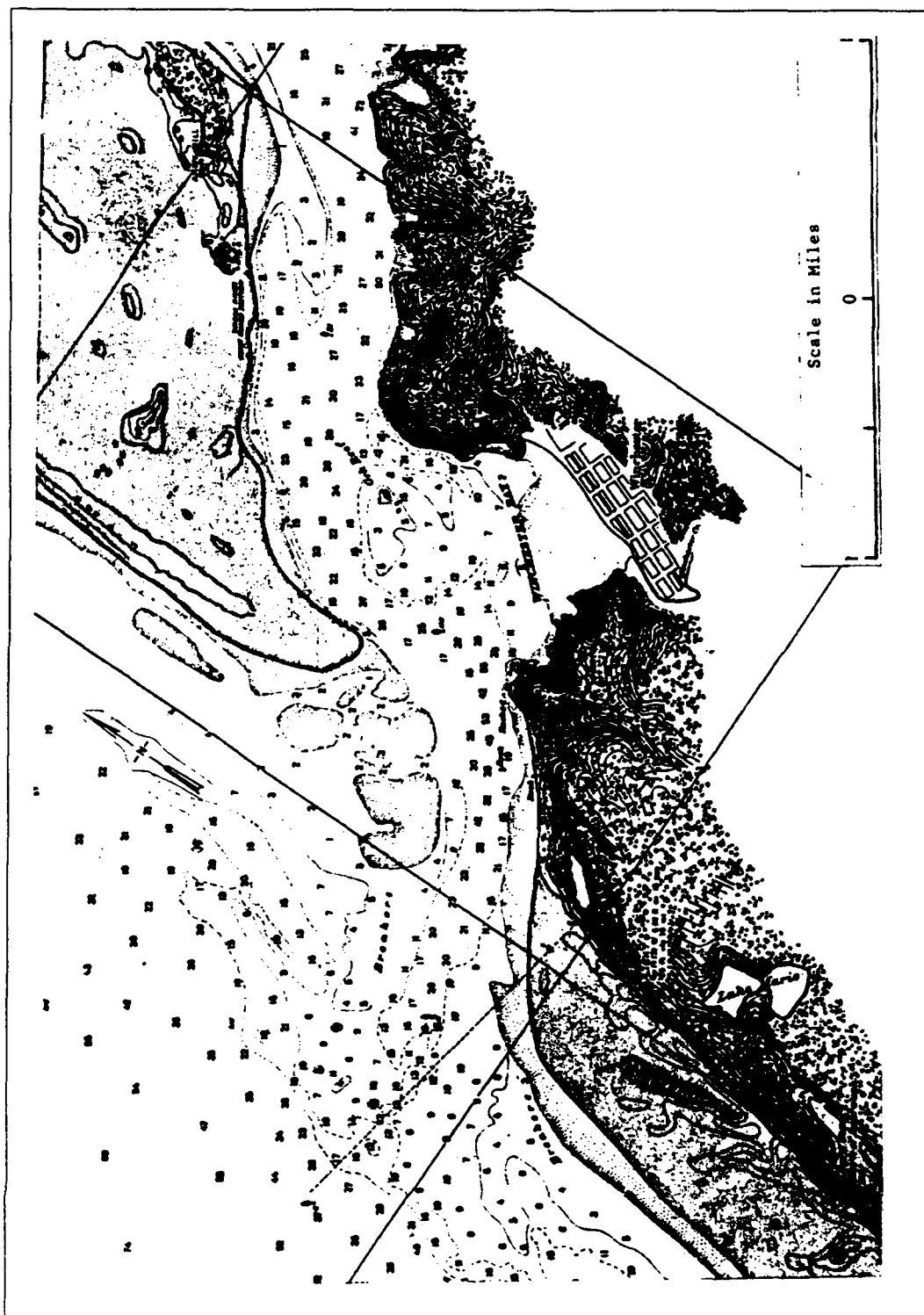
Figure 3. River mouth prior to improvements, 1917
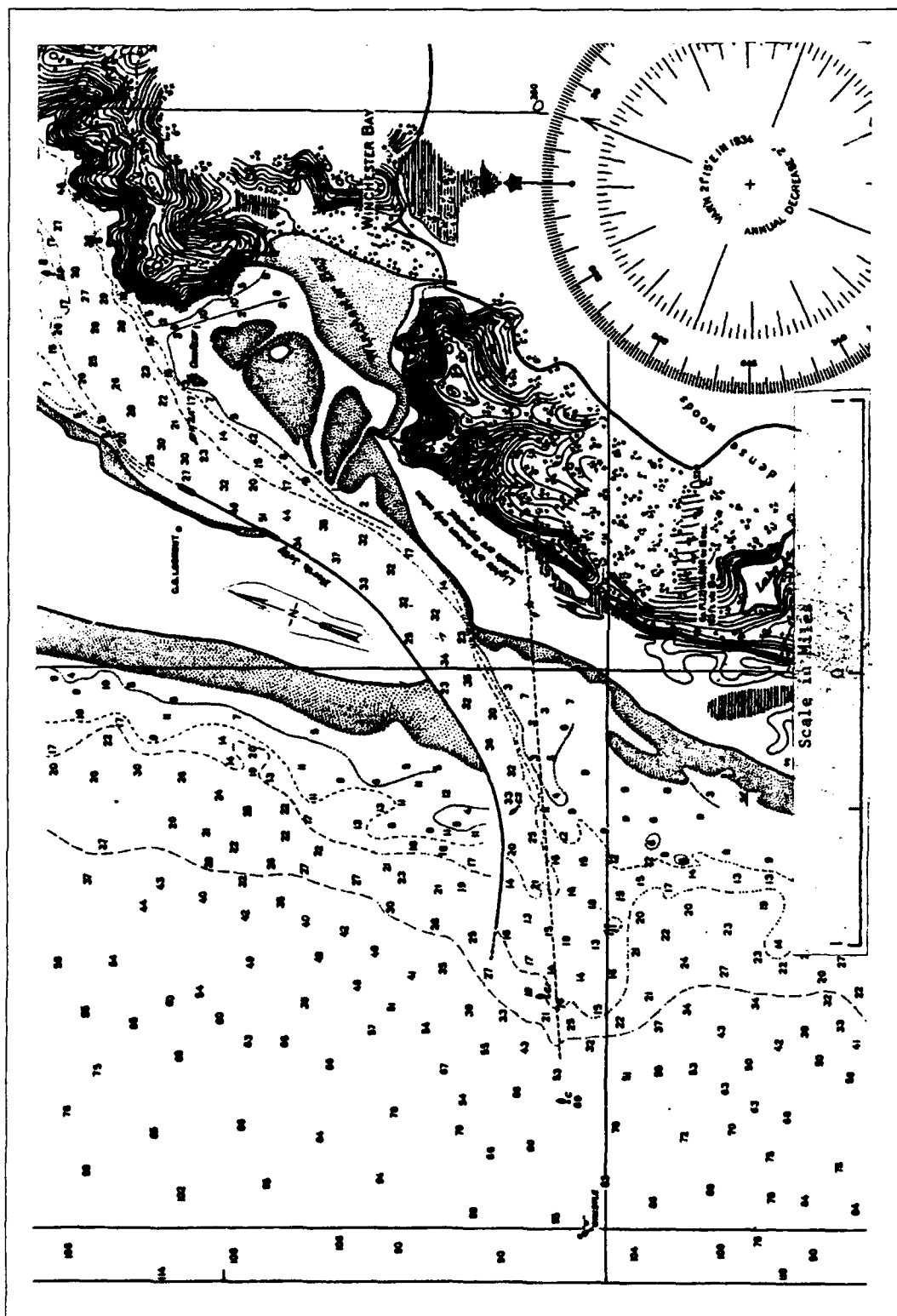
Figure 4. River mouth after construction of north jetty, 1932
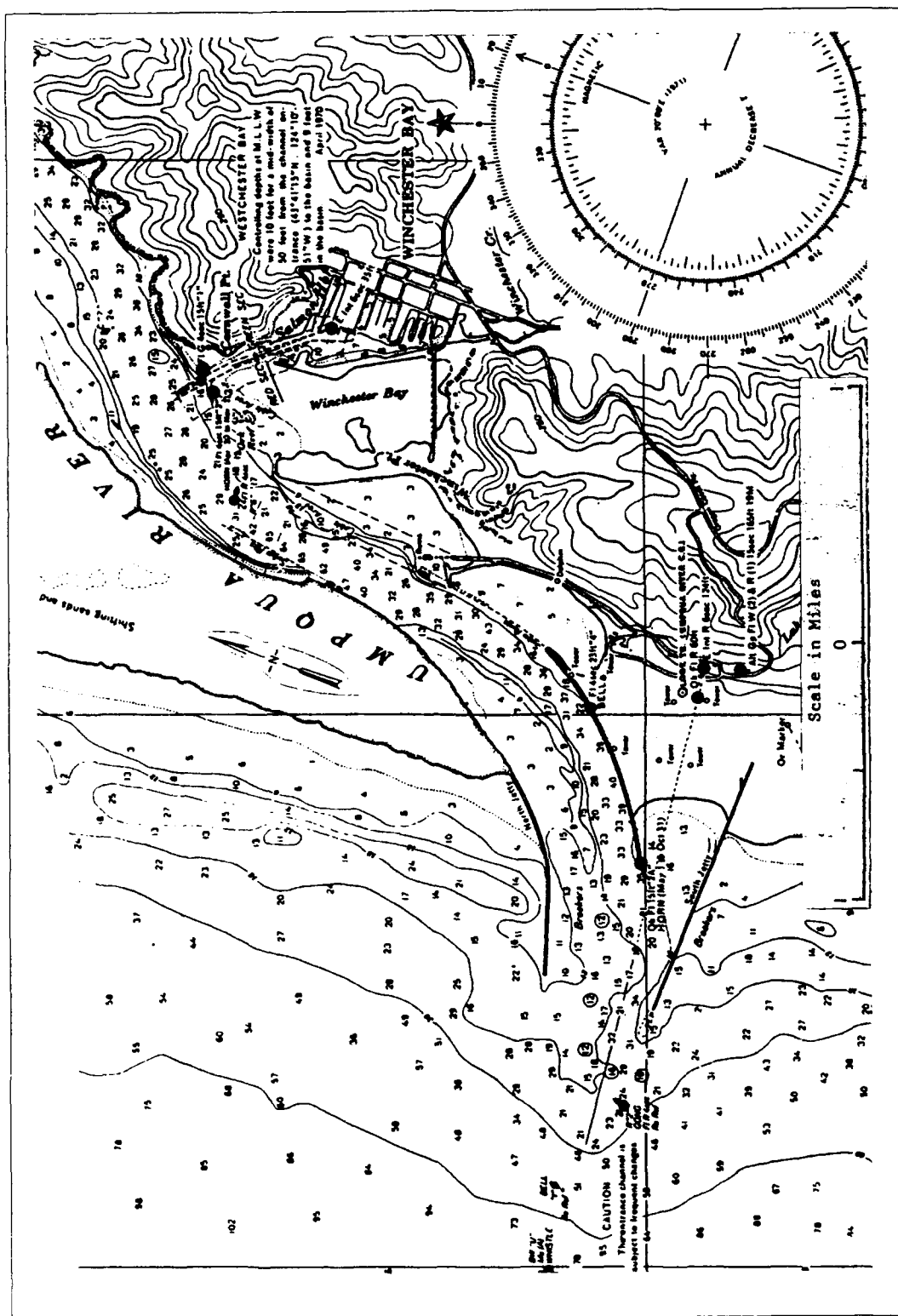
Figure 5. River mouth after construction of north and south jetties and the initial training jetty, 1970

Figure 6. Waves breaking between jetties at Umpqua River entrance, 1966
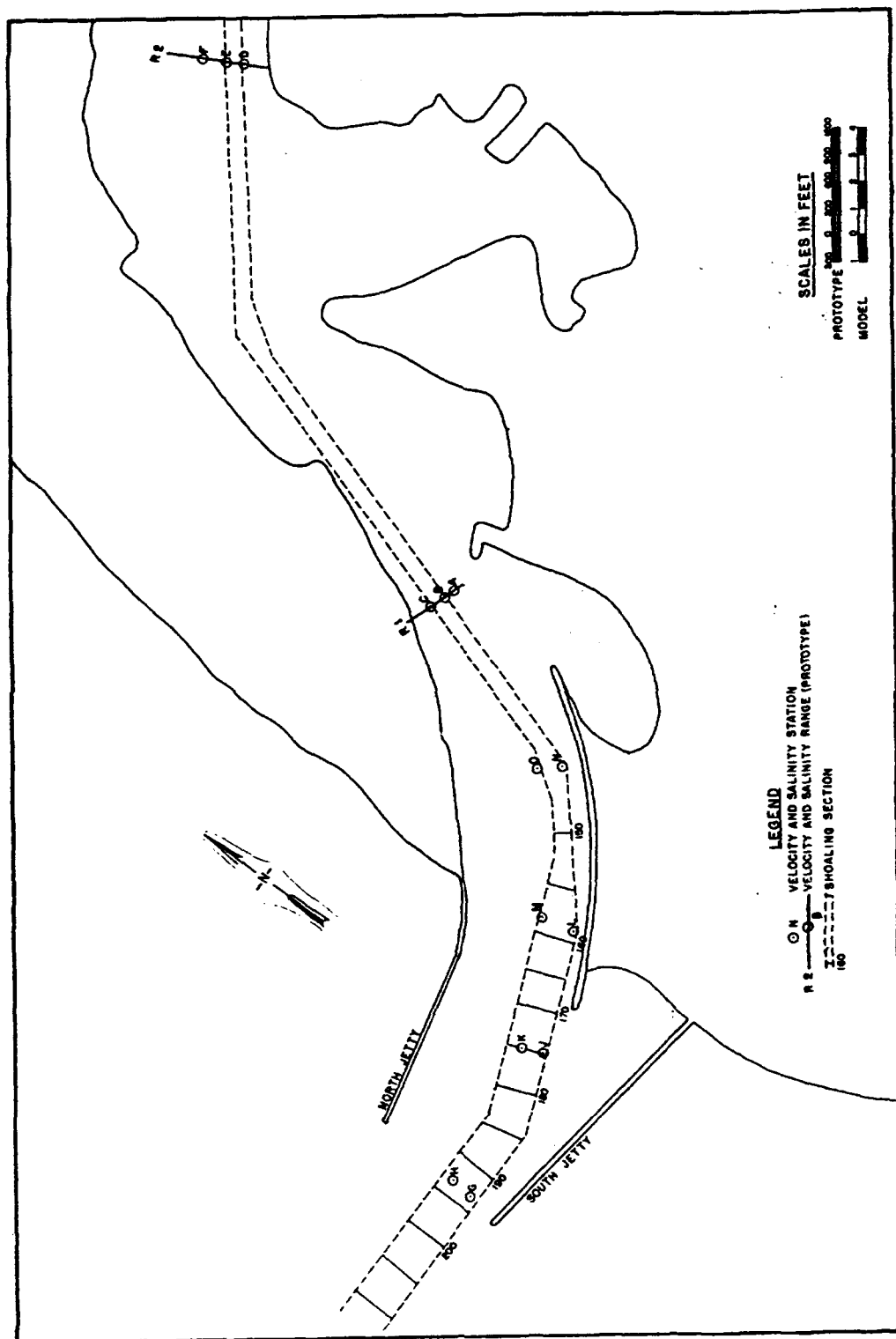
Figure 7. Stations and ranges used for collection of current and salinity data in the 1970 model investigation
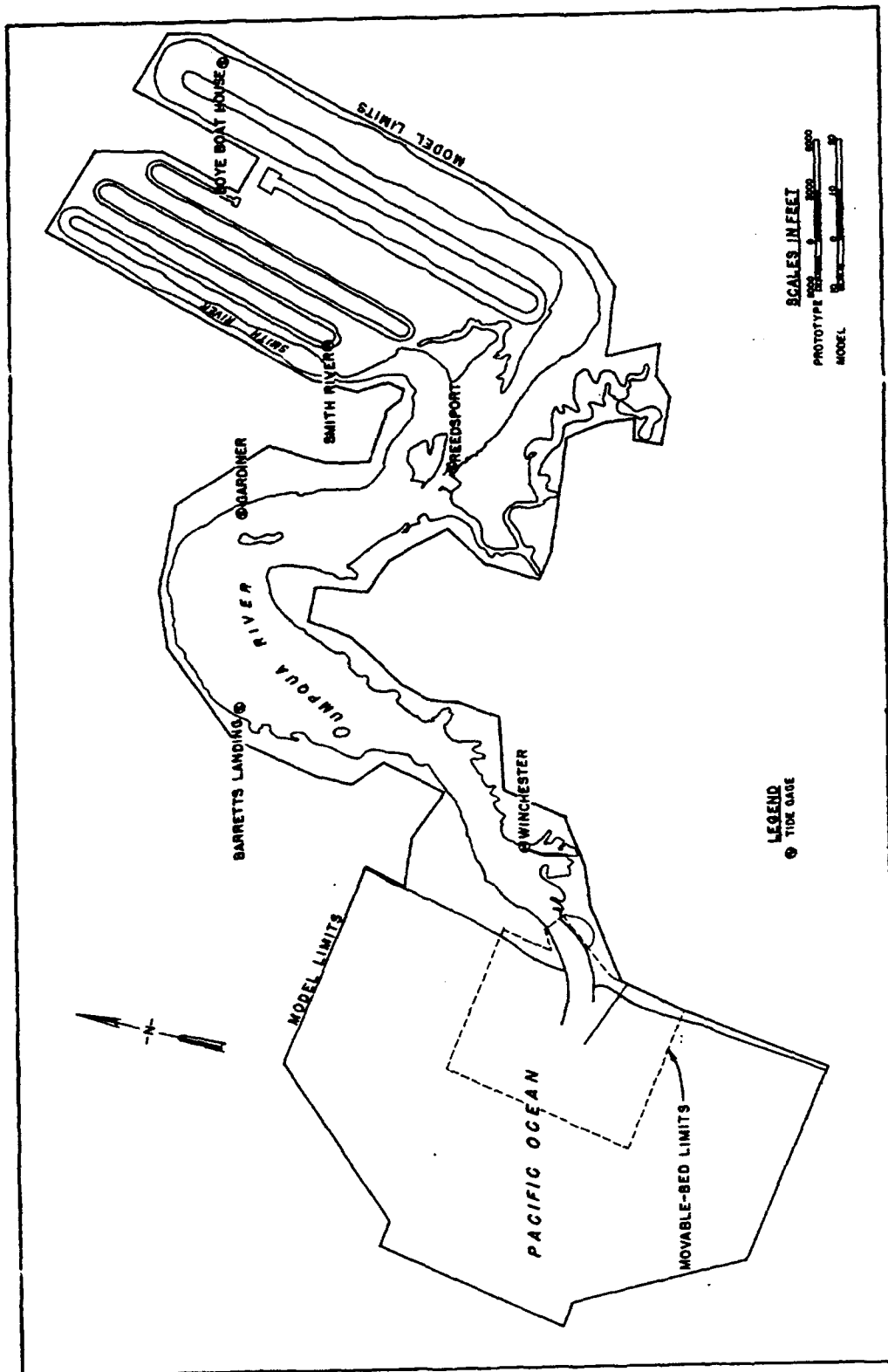
Figure 8. Location of tide stations in the 1970 model investigation
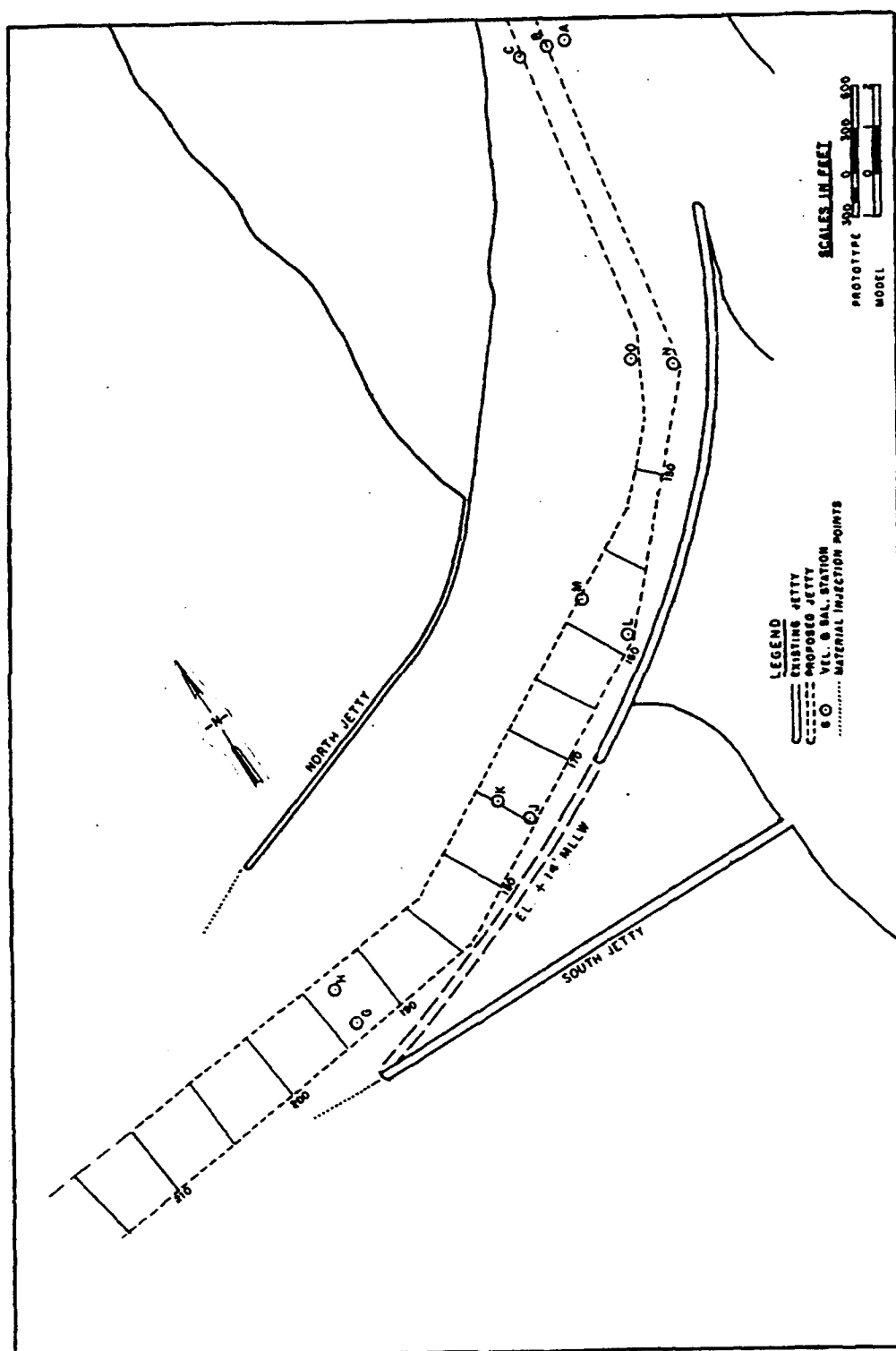
Figure 9. View of training jetty extension recommended in the 1970 model investigation

Figure 10. Location of velocity and salinity stations, wave gages, velocity meters, and tide gage for prototype data collection
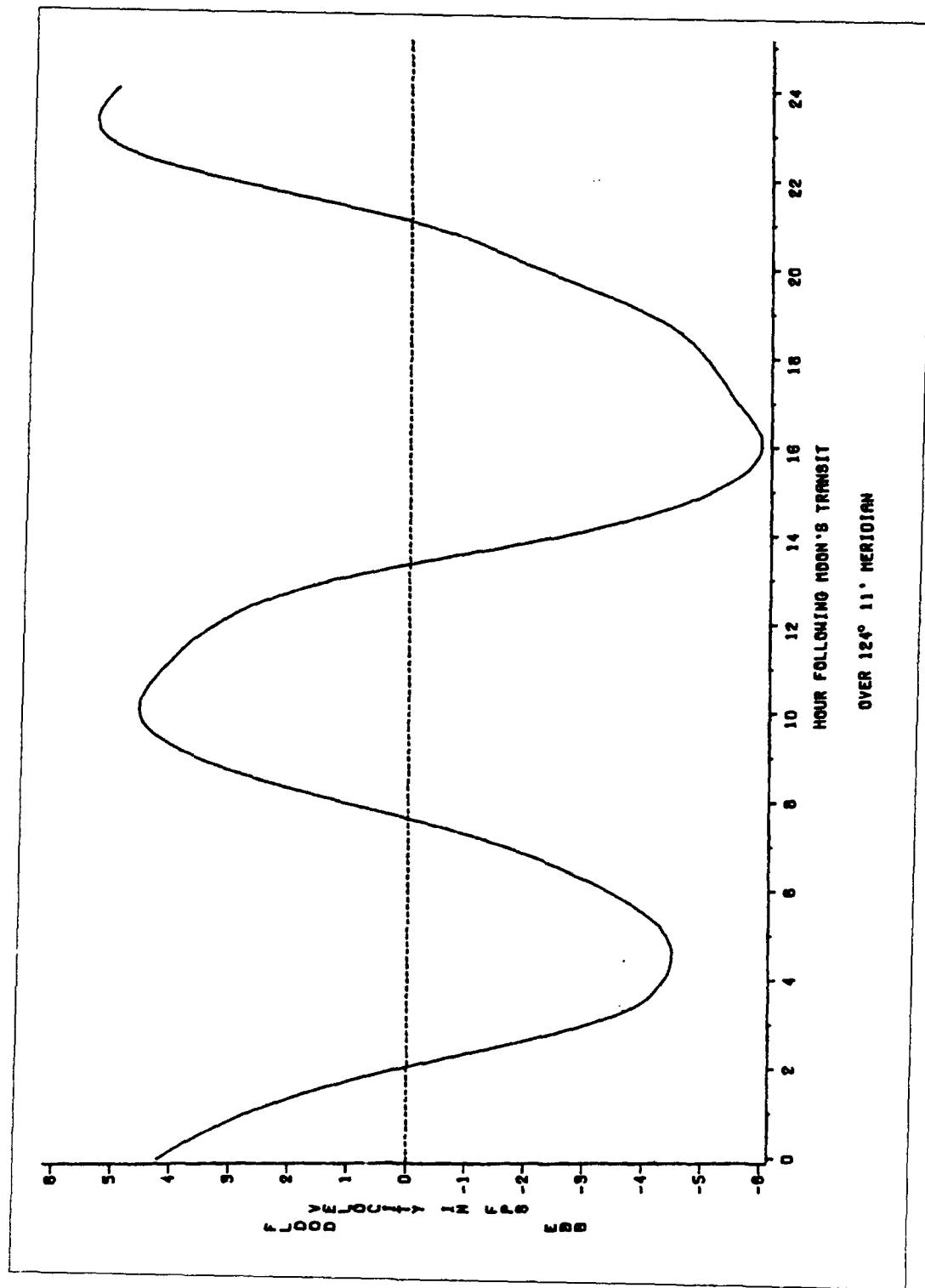
Figure 11. Prototype velocity data at station A, surface depth, August 1983
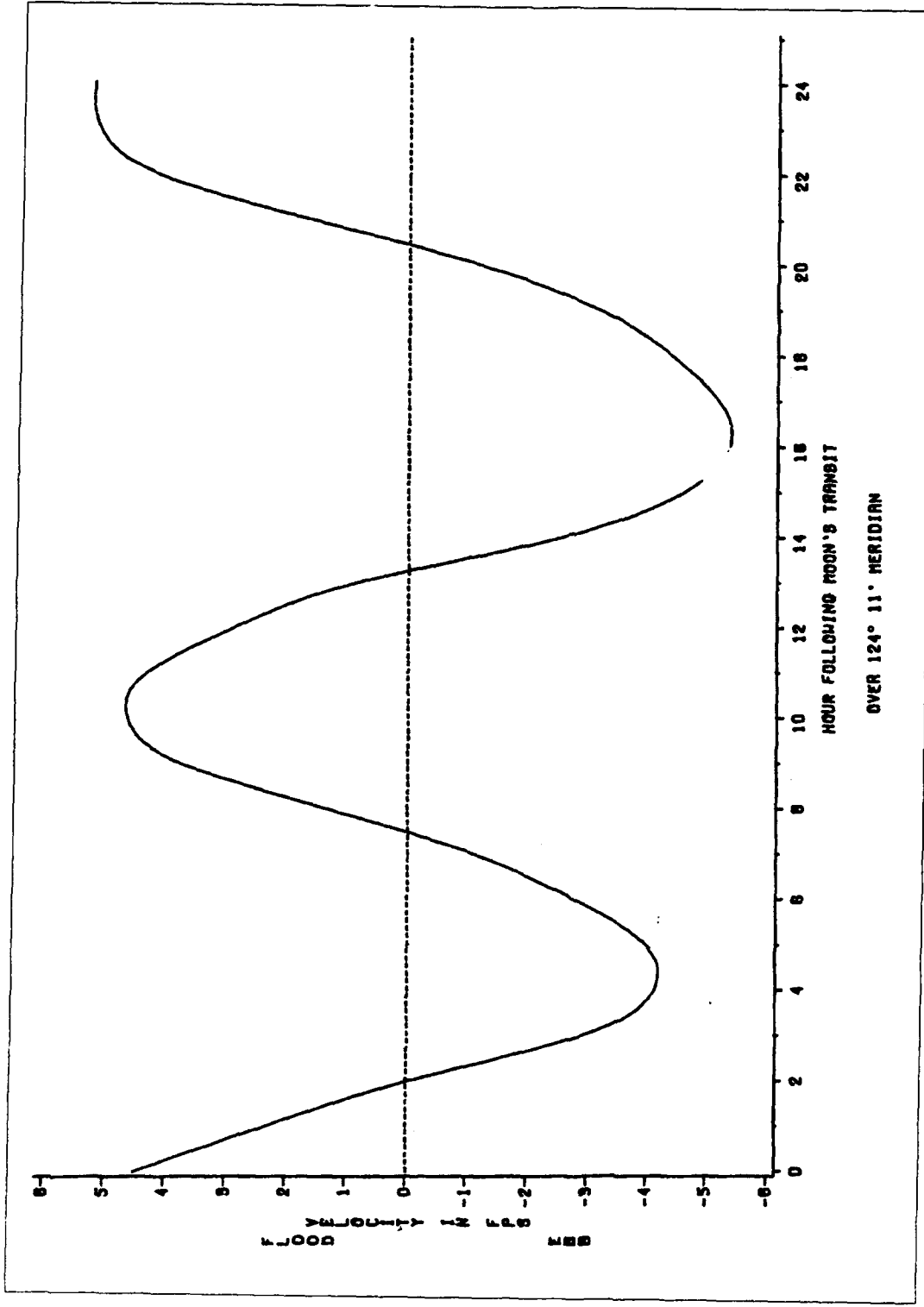
Figure 12. Prototype velocity data at station A, mid-depth, August 1983
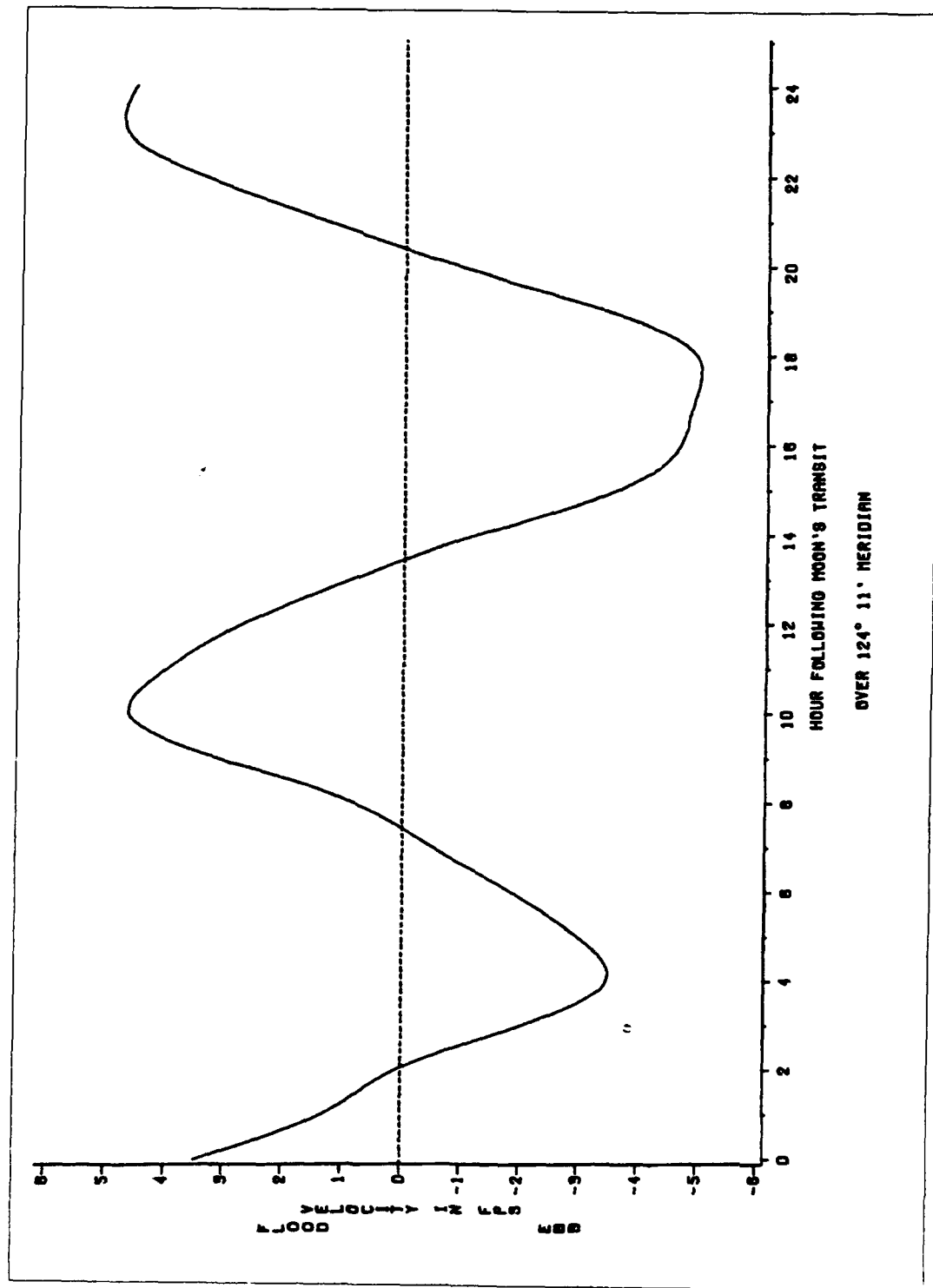
Figure 13. Prototype velocity data at station A, bottom depth, August 1983
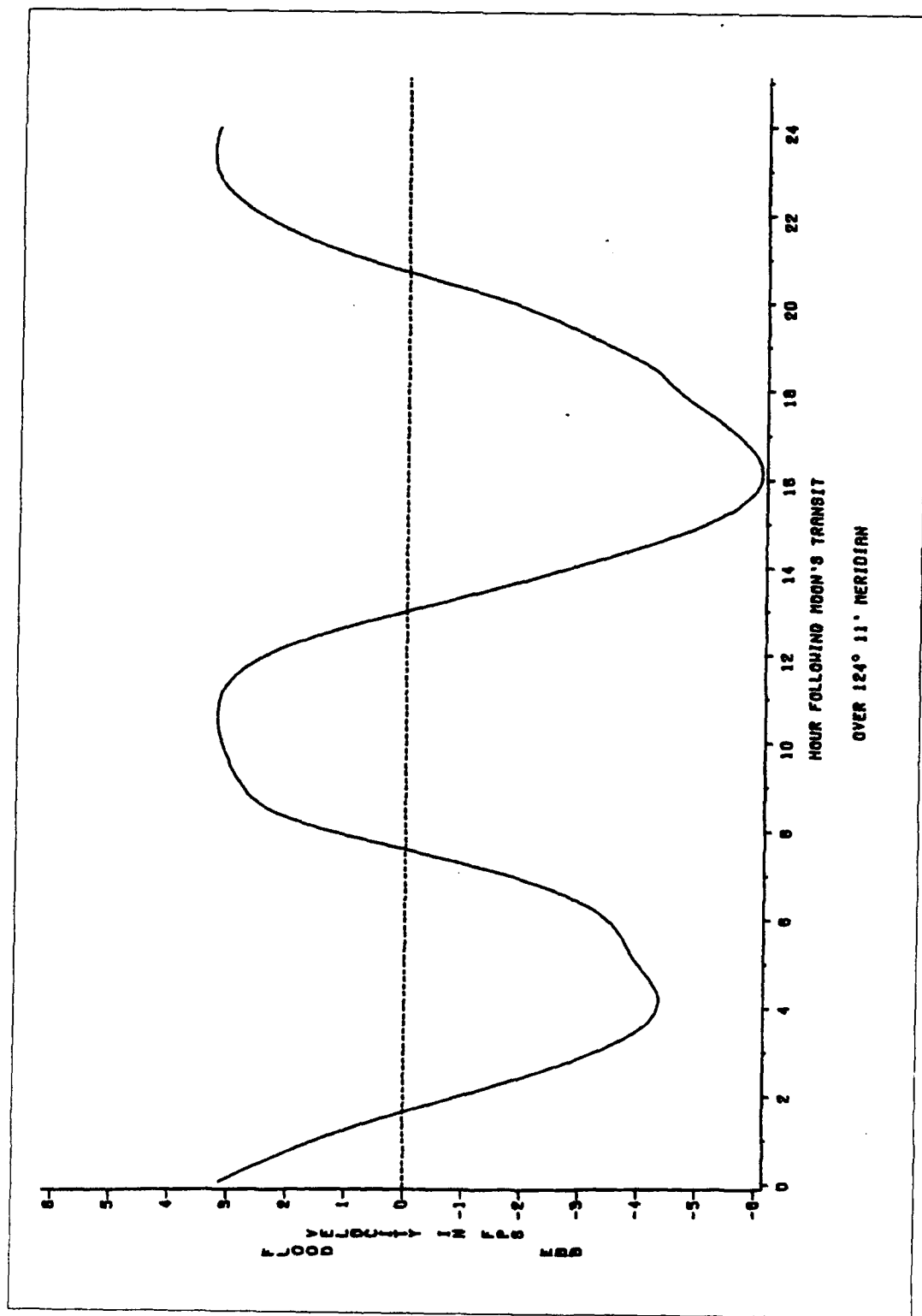
Figure 14. Prototype velocity data at station B, surface depth, August 1983
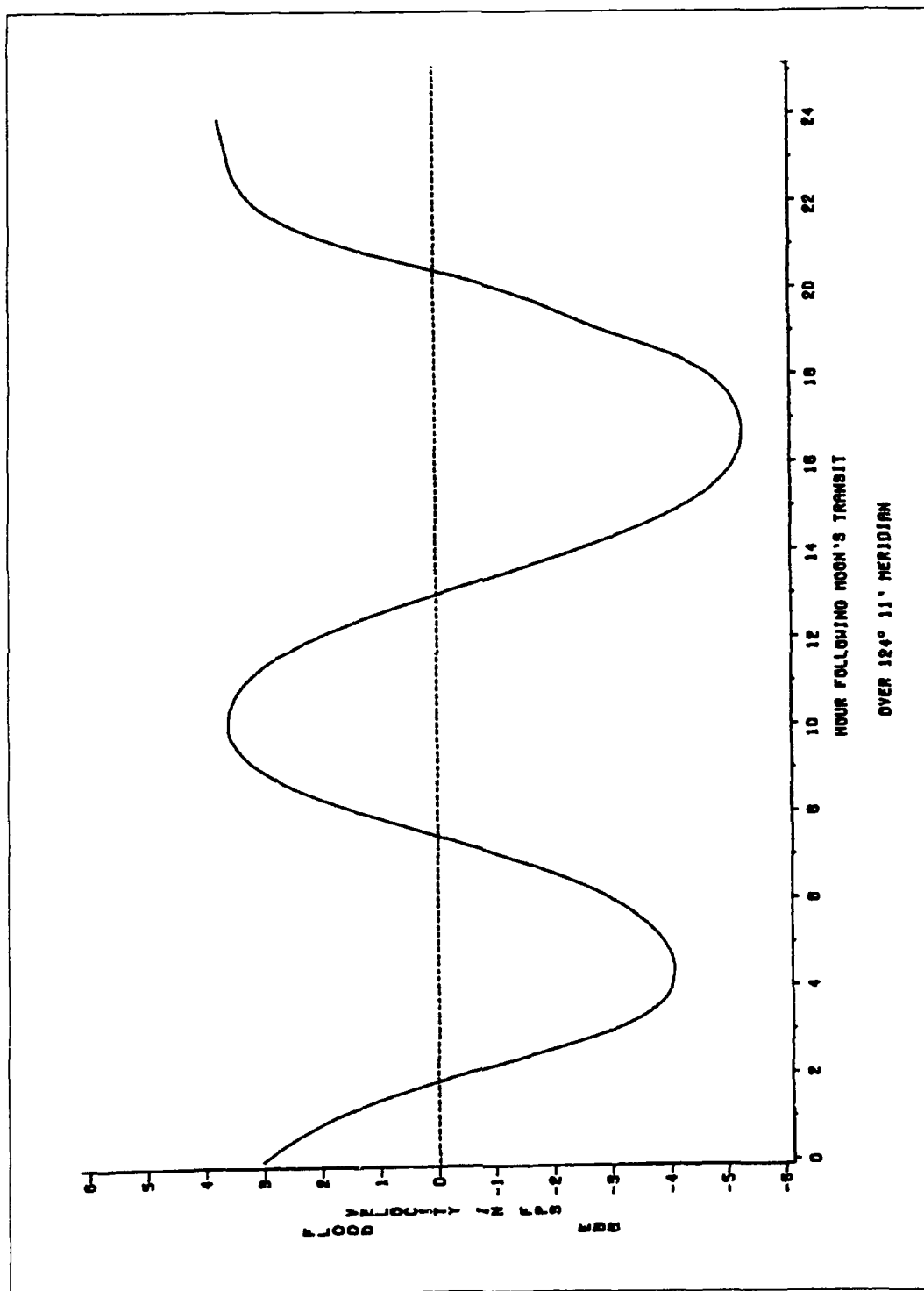
Figure 15. Prototype velocity data at station B, mid-depth, August 1983
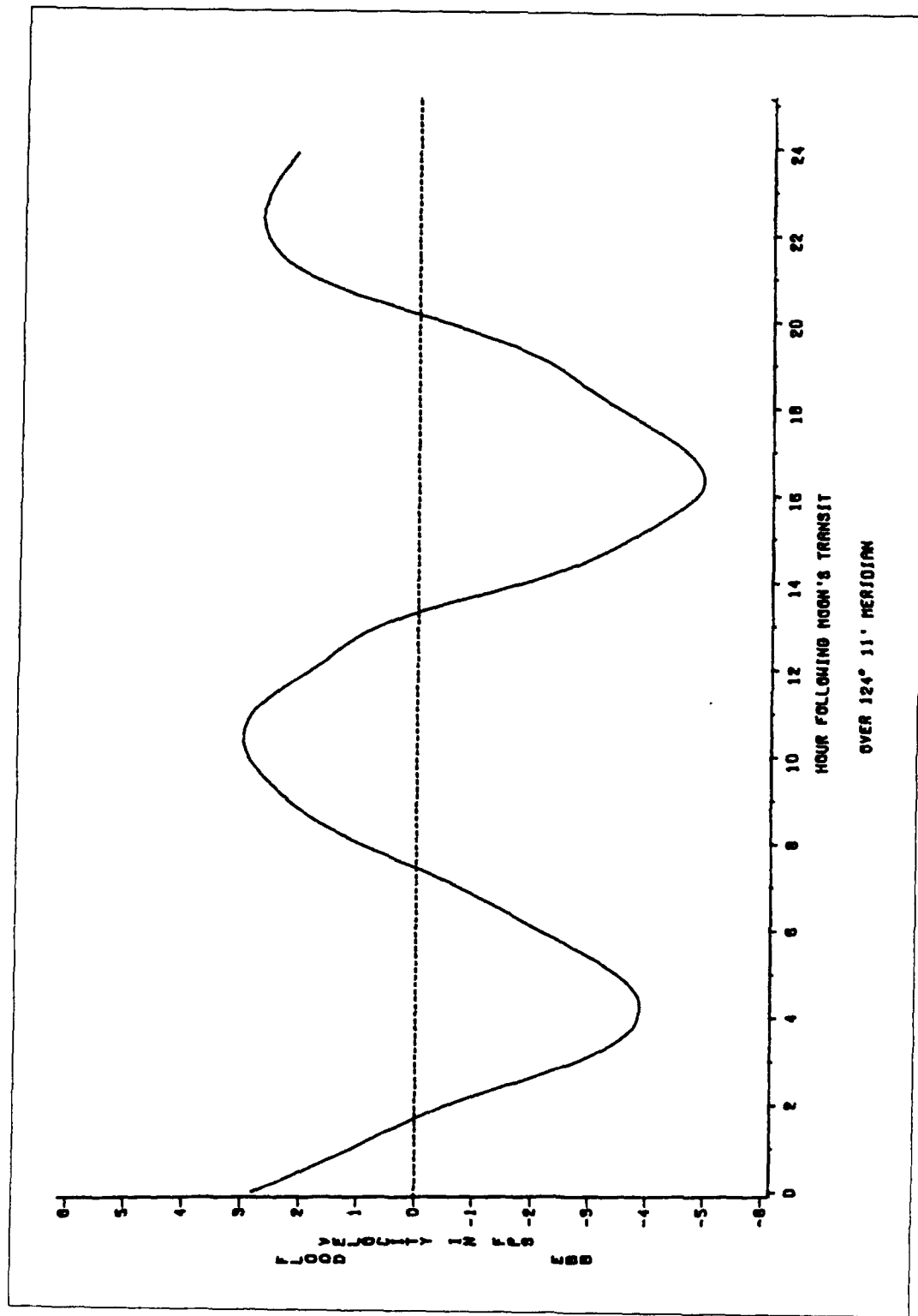
Figure 16. Prototype data at station B, bottom depth, August 1983
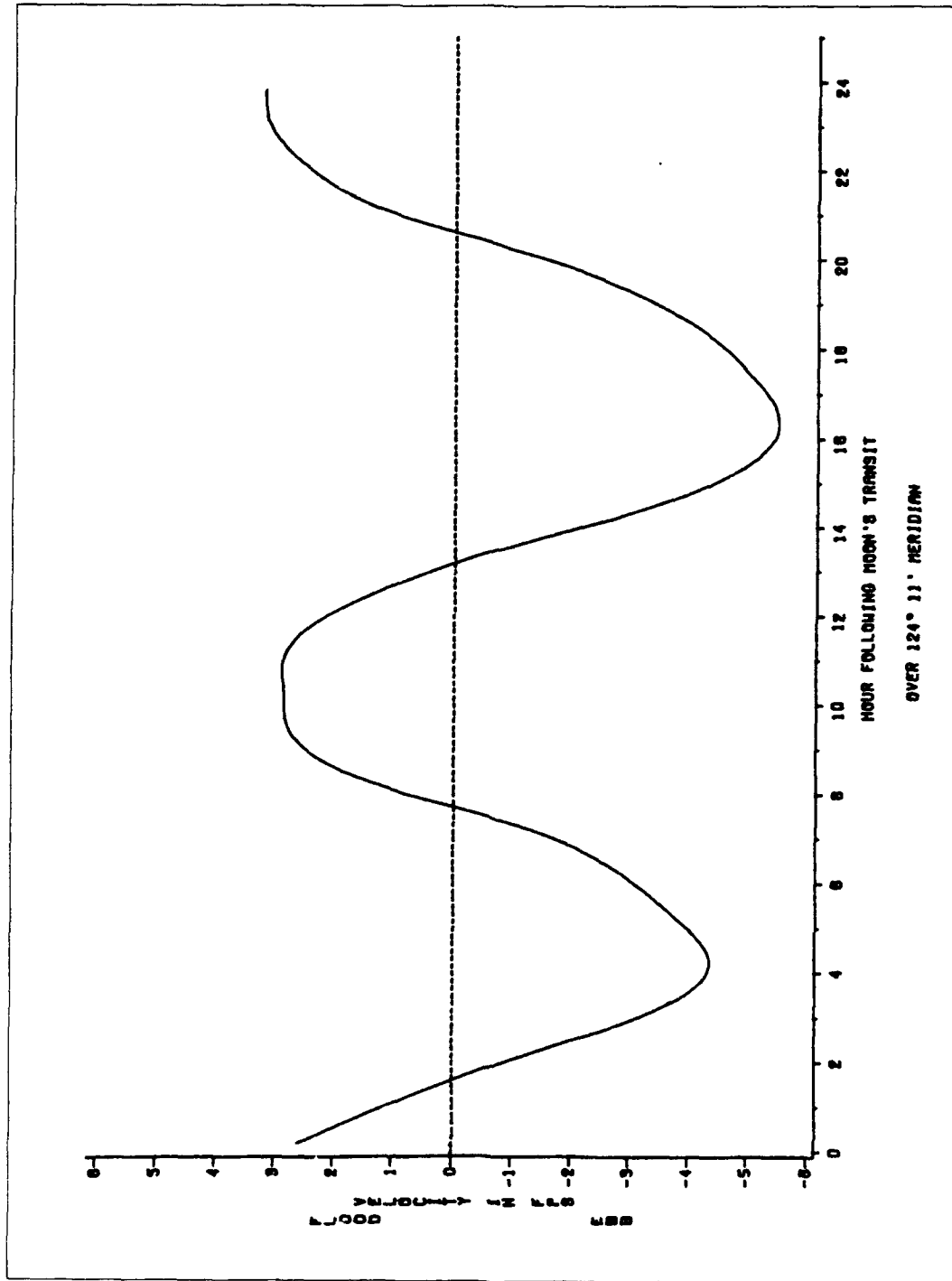
Figure 17. Prototype velocity data at station C, surface depth, August 1983
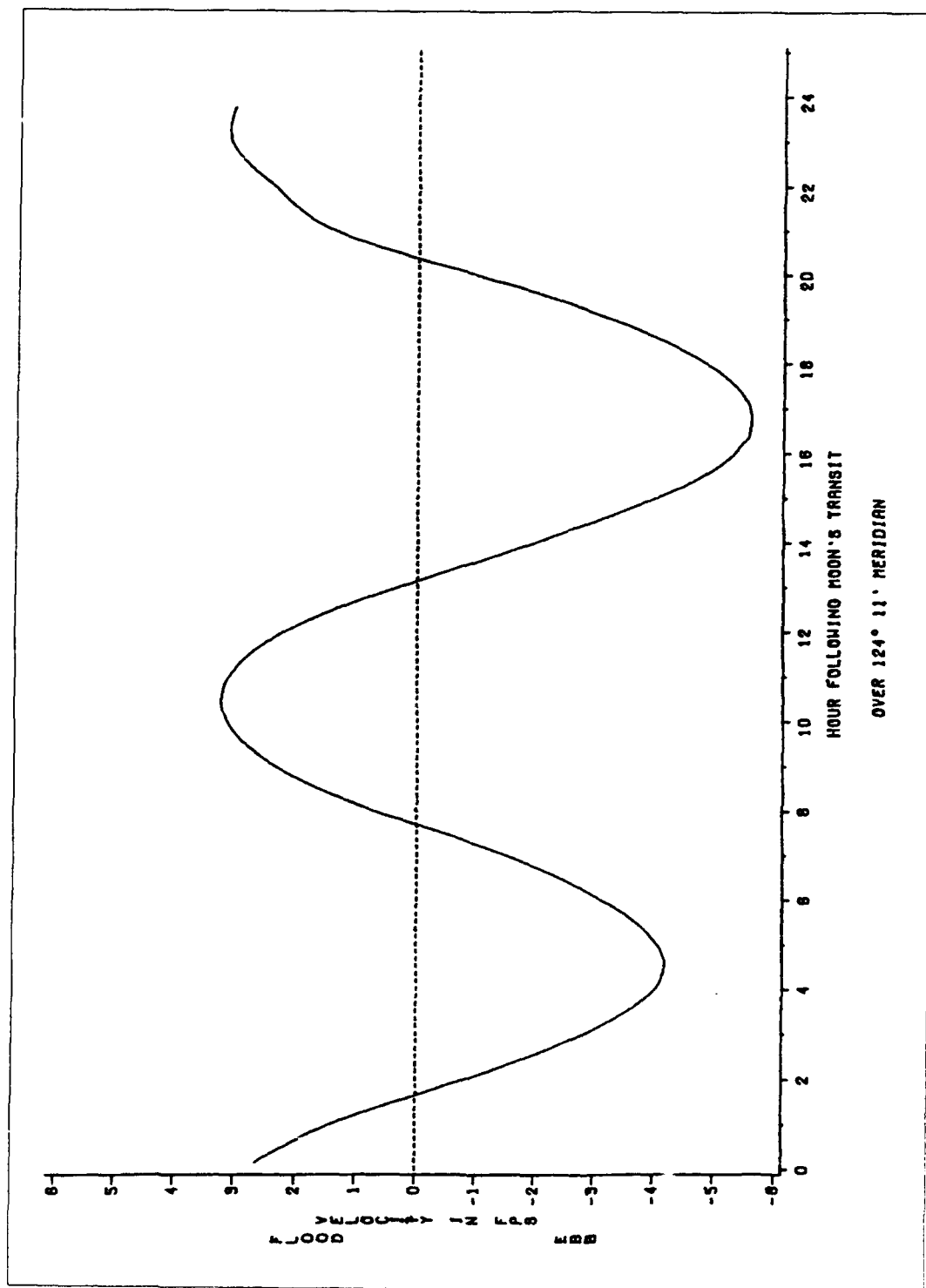
Figure 18. Prototype velocity data at station C, mid-depth, August 1983
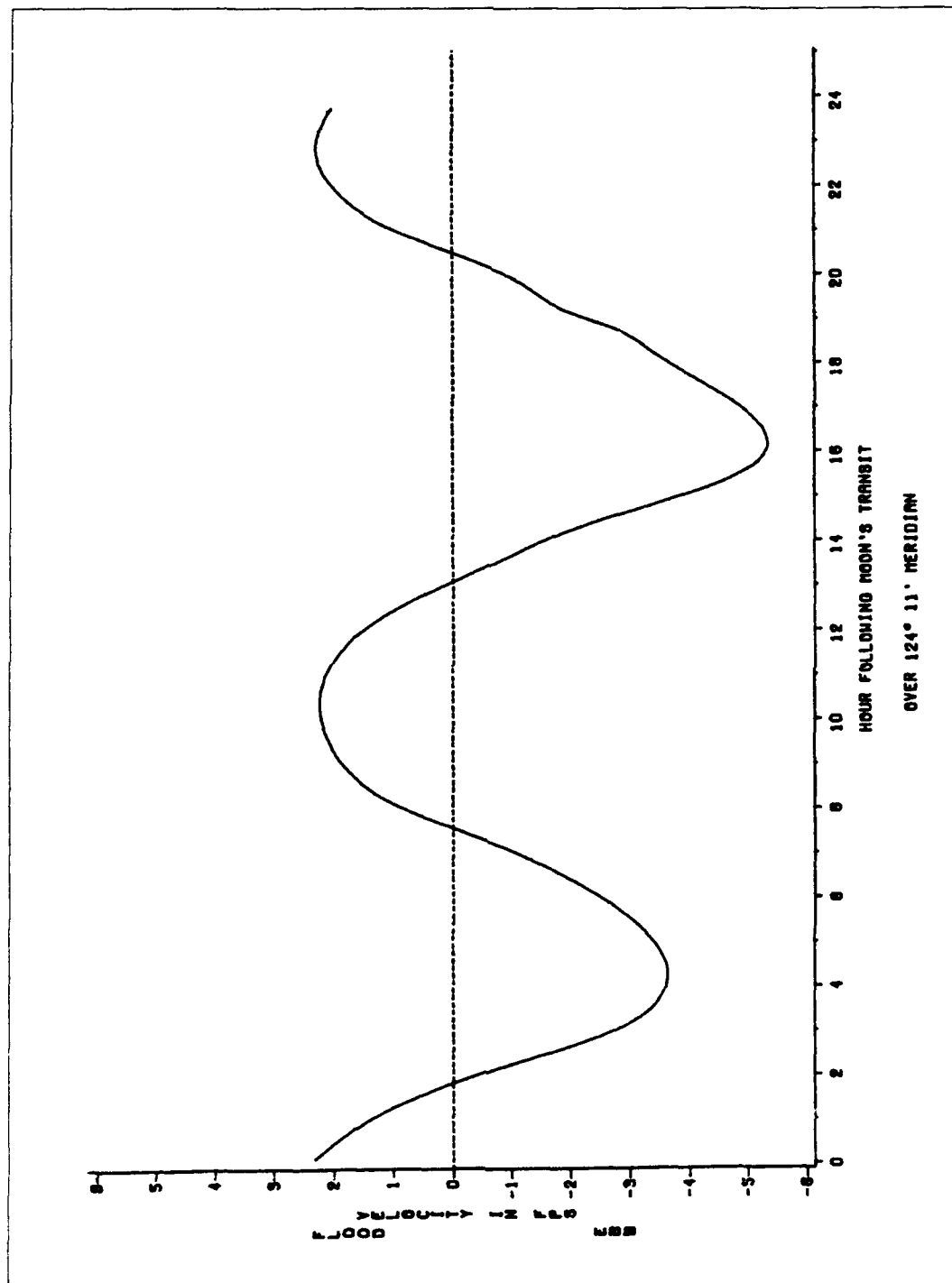
Figure 19. Prototype velocity data at station C, bottom depth, August 1983
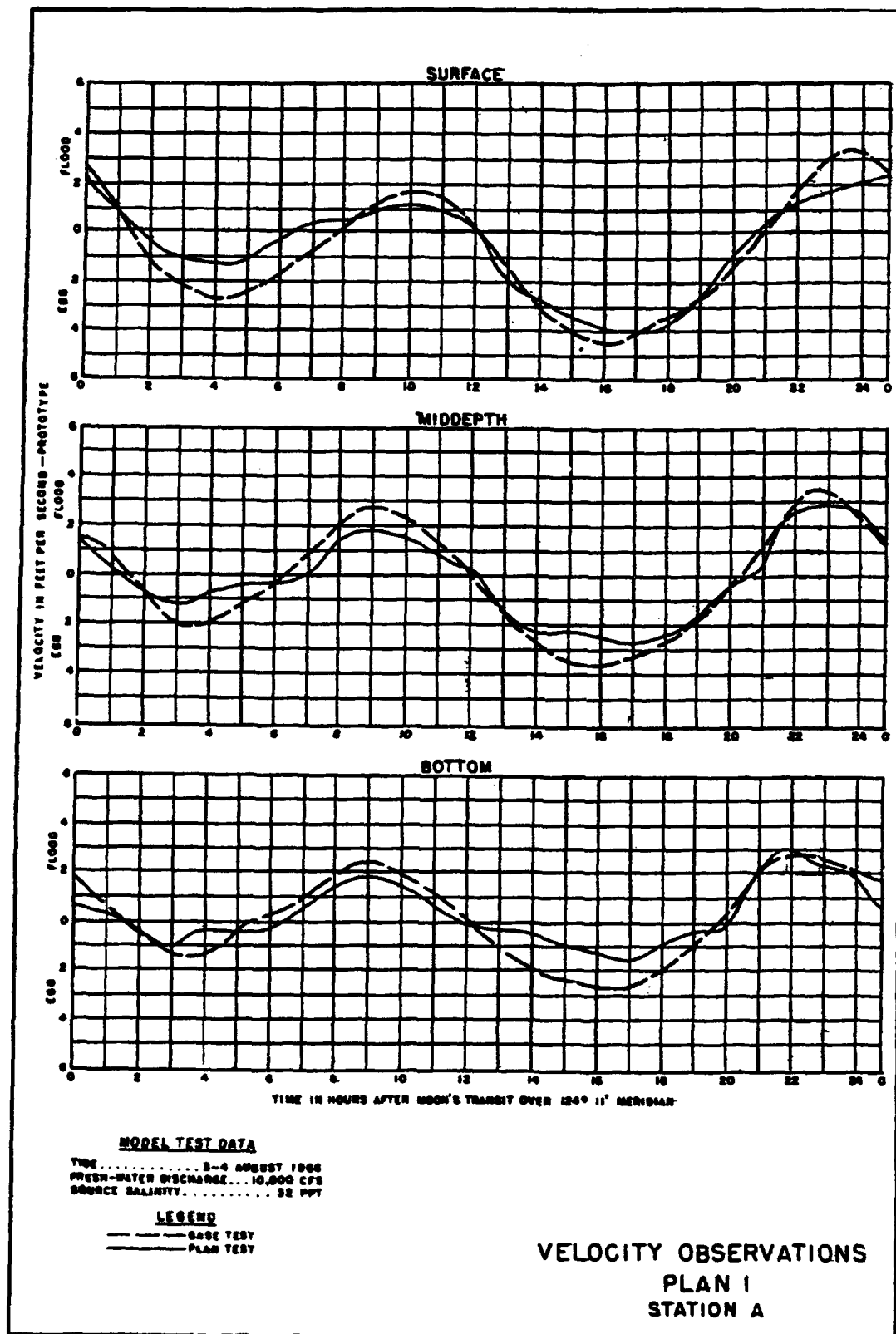
Figure 20. Model velocity data at station A
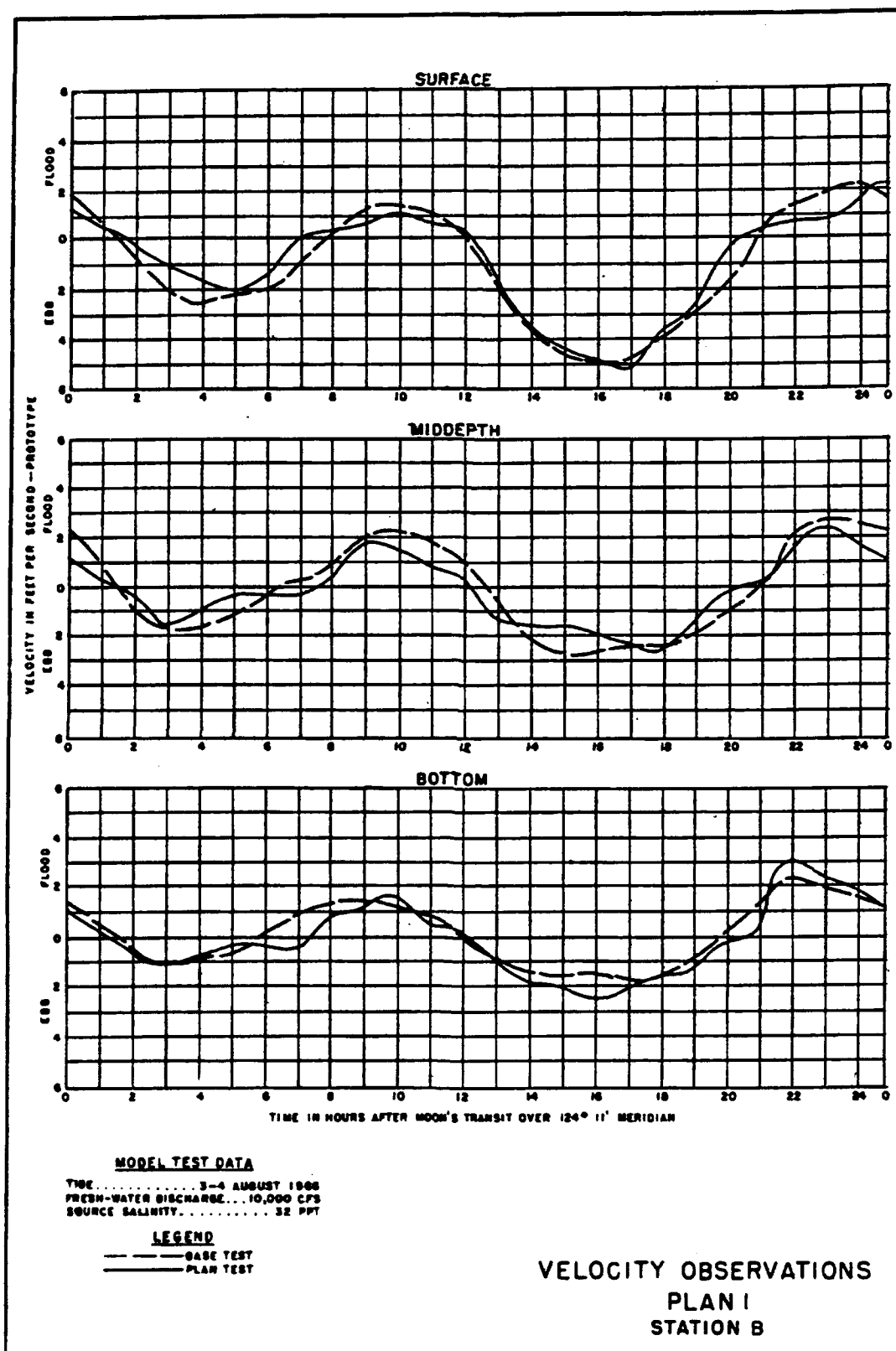
Figure 21. Model velocity data at station B
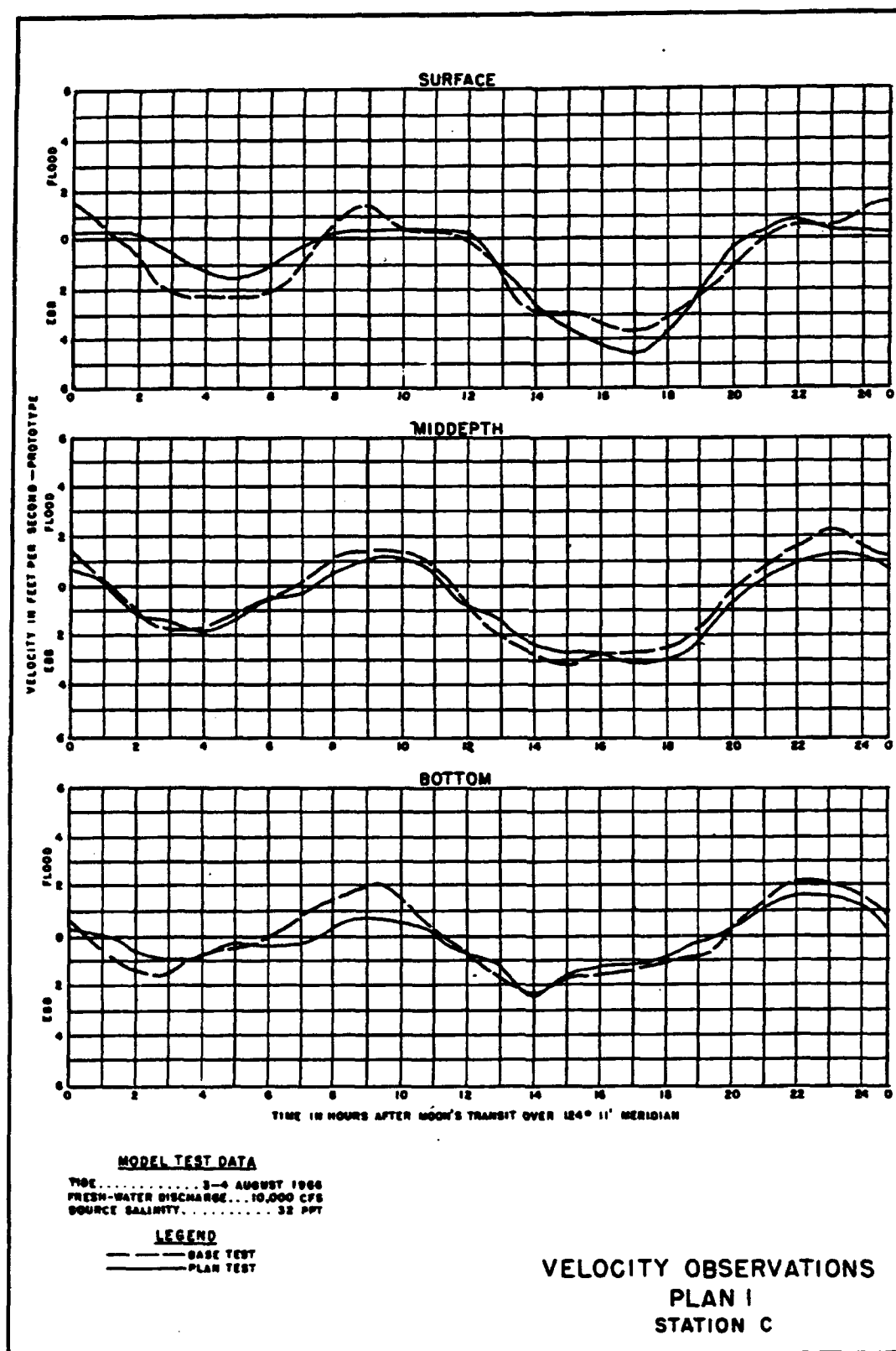
Figure 22. Model velocity data at station C
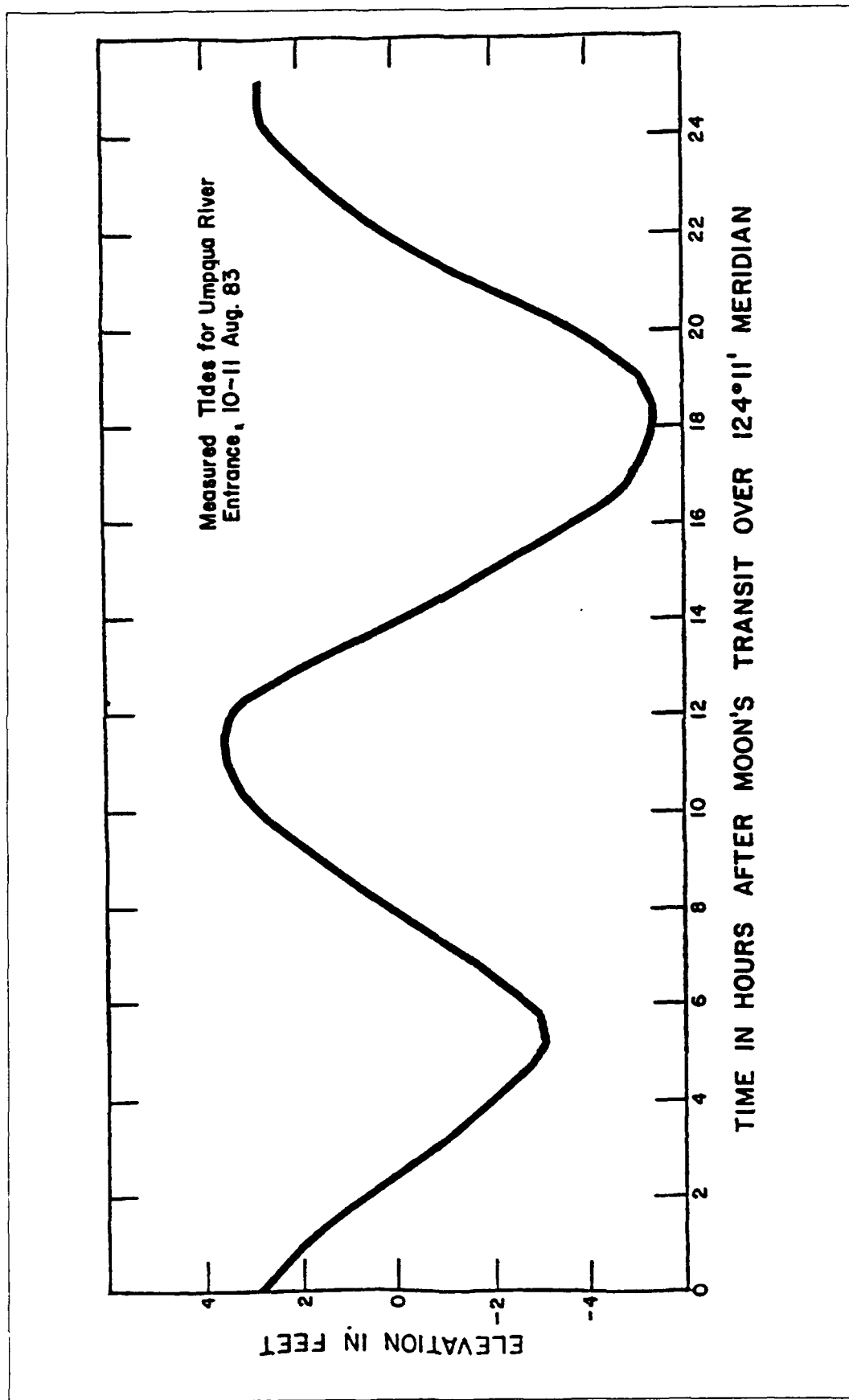
Figure 23. Tidal range during prototype velocity measurements, 1983

The following text appears within the figure:

Measured Tides for Umpqua River
Entrance, IO–II Aug. 83

ELEVATION IN FEET

TIME IN HOURS AFTER MOON'S TRANSIT OVER 124°II' MERIDIAN
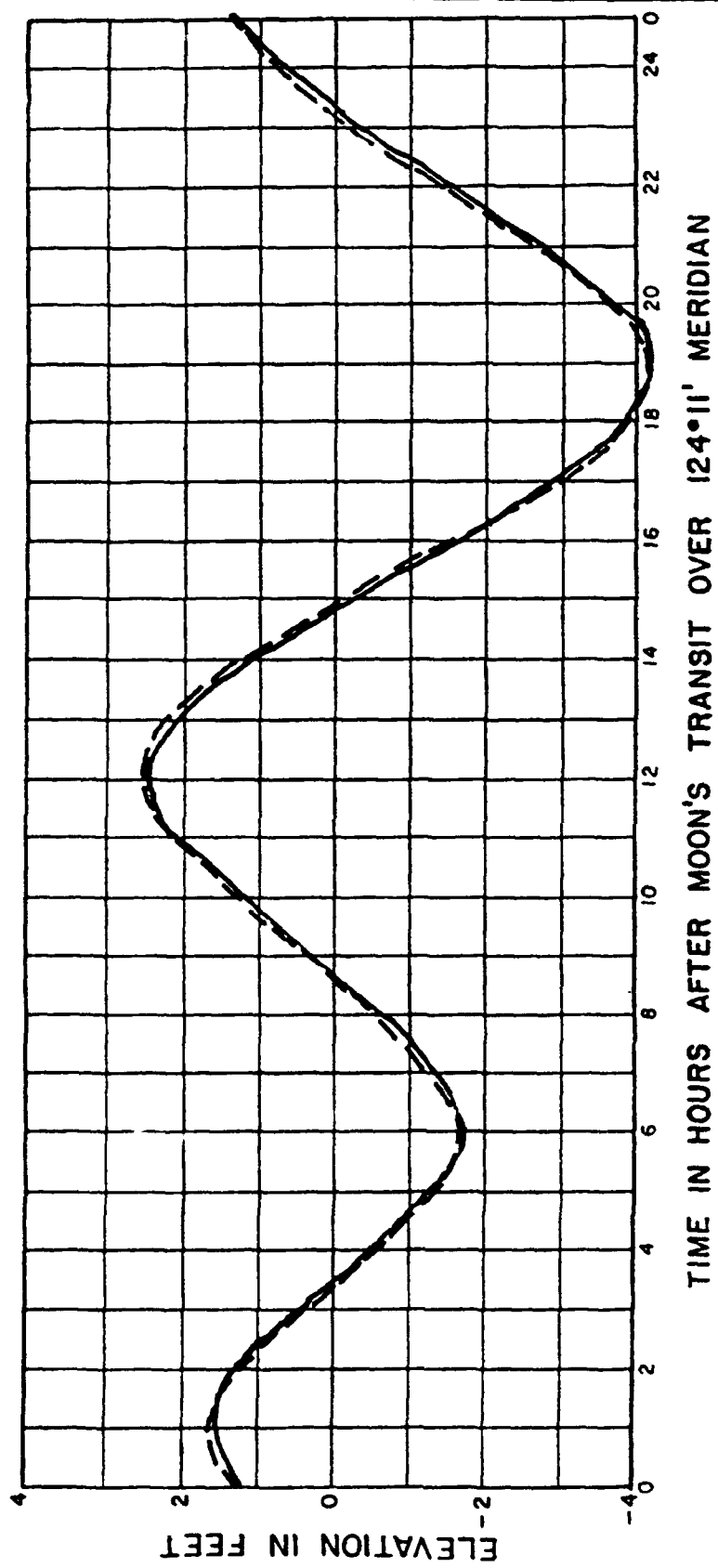
Figure 24. Tidal range used during model testing

Figure 25. Current patterns obtained in the model for flood tidal flow conditions

Figure 26. Current patterns obtained in the model for ebb tidal flow conditions
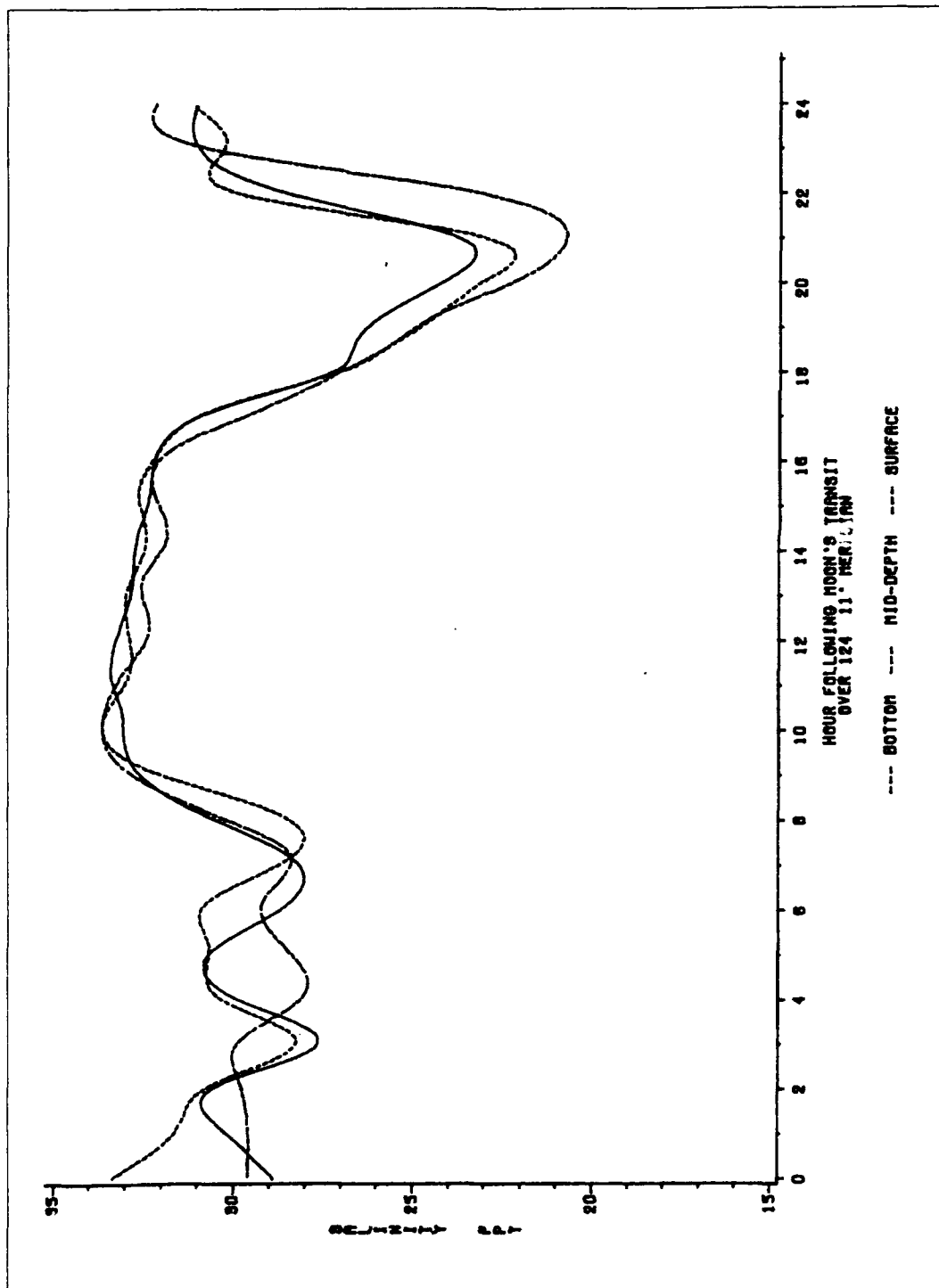
Figure 27. Results of salinity samples obtained at station B, August 1983
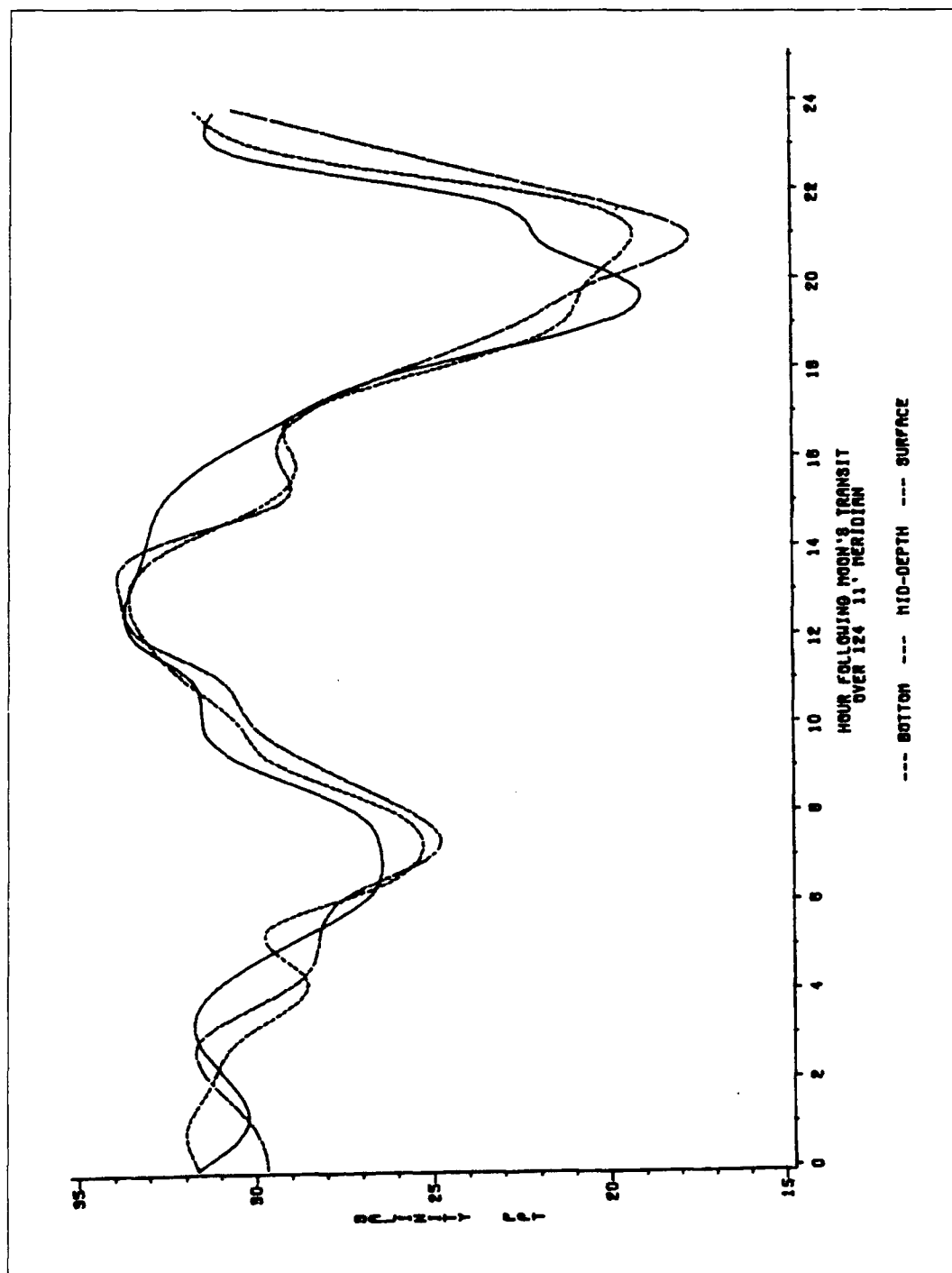
Figure 28. Results of salinity samples obtained at station D, August 1983
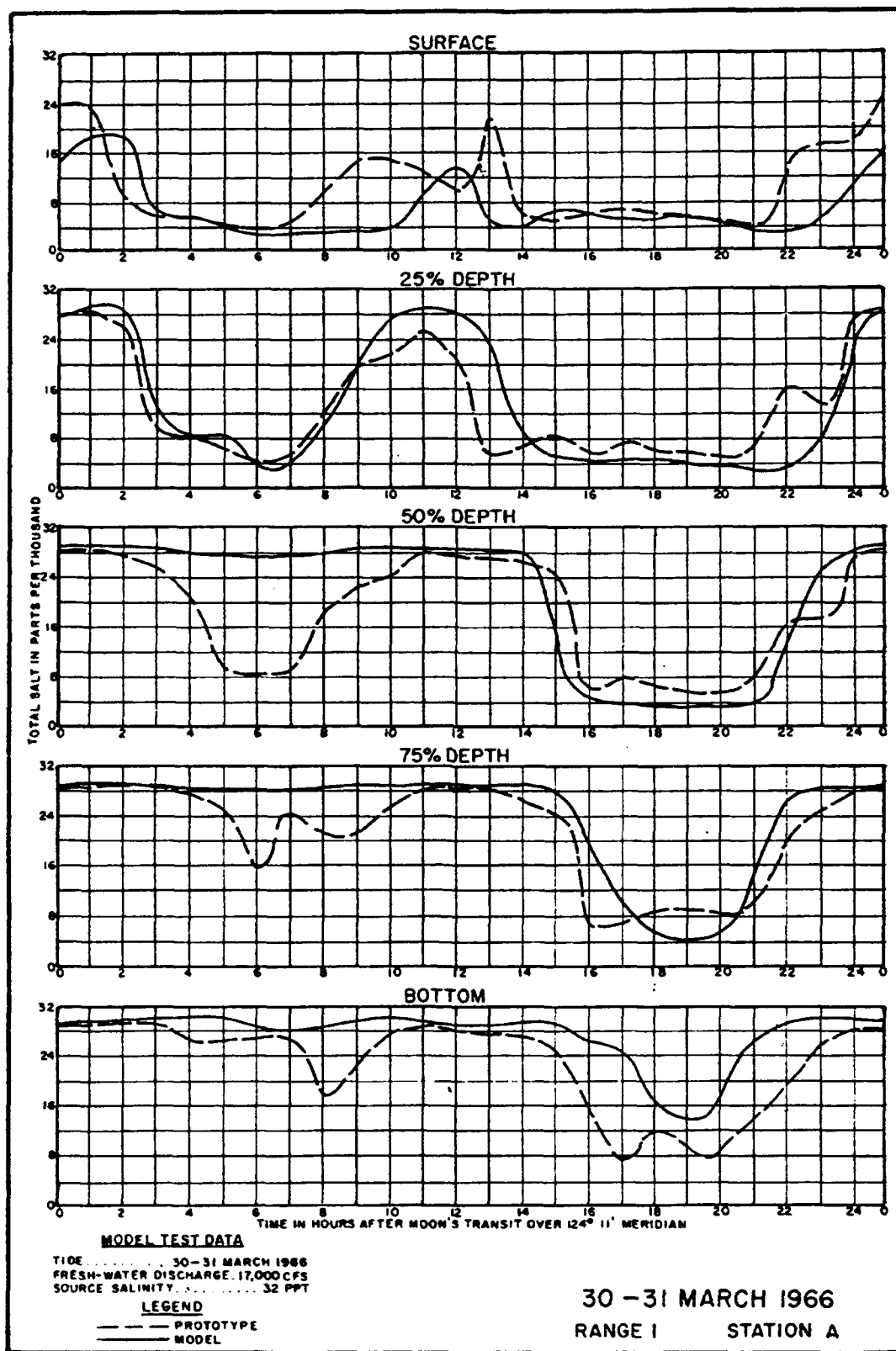
Figure 29. Comparison of the model versus prototype data obtained in 1966
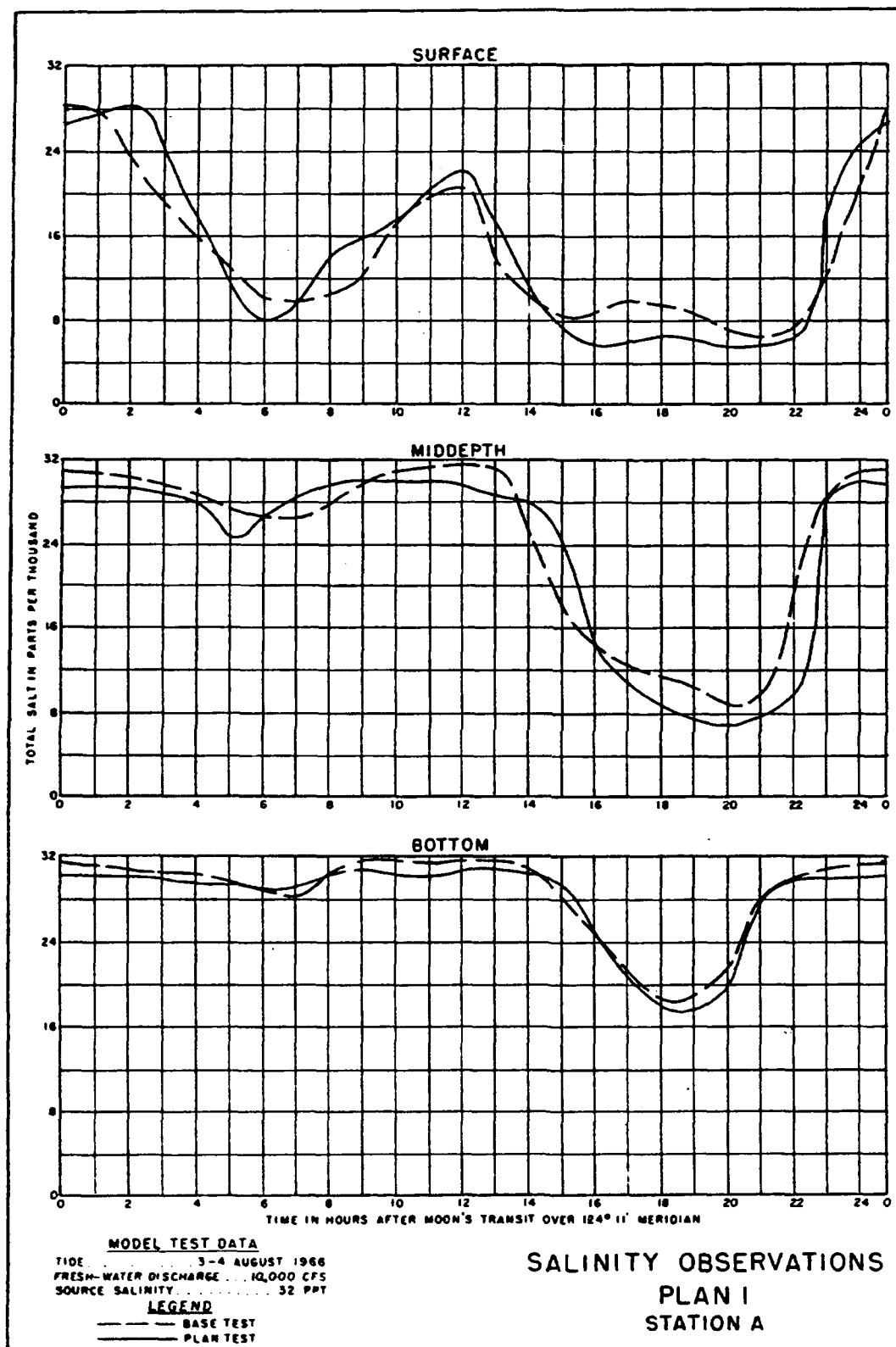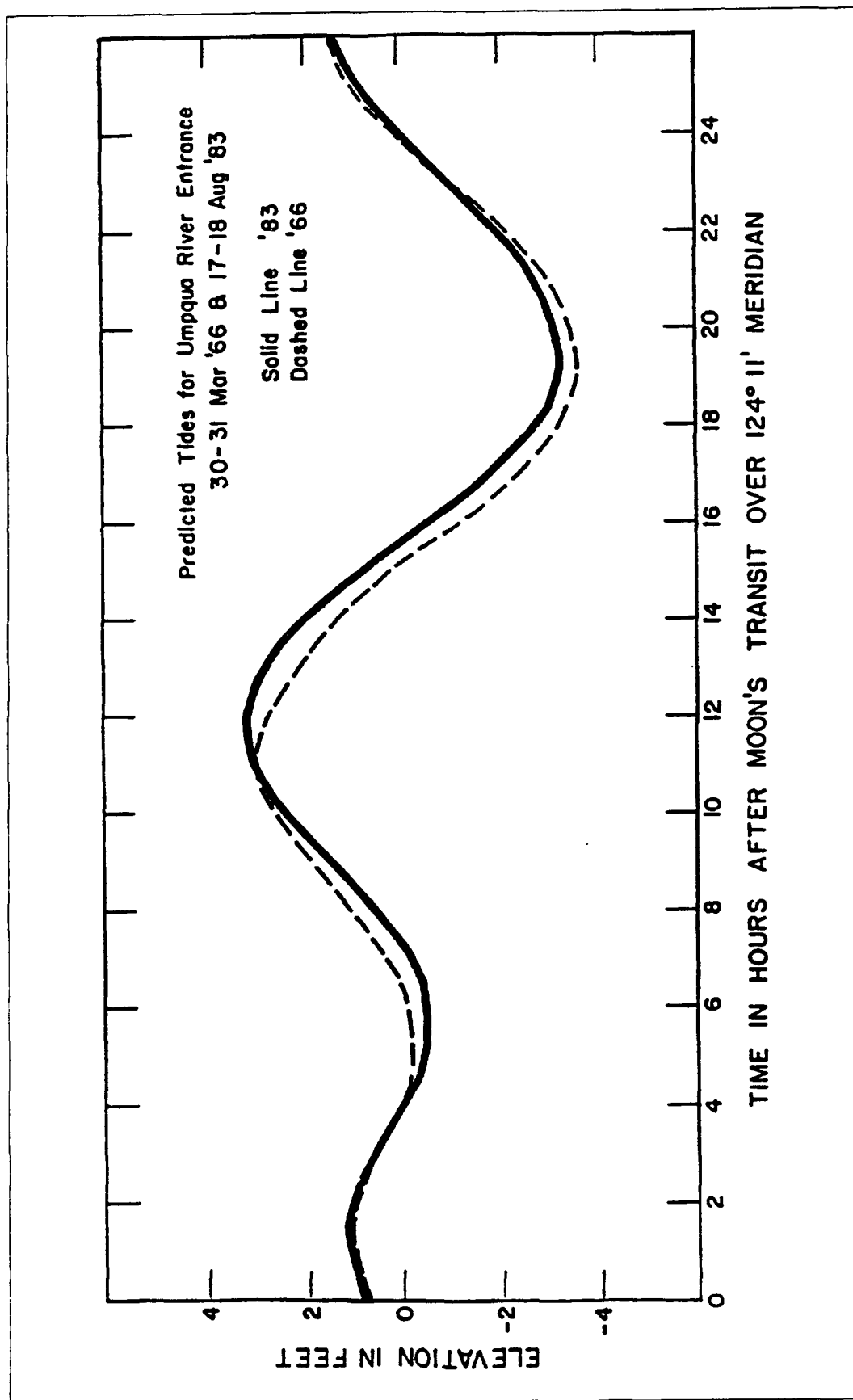
Figure 30. Comparison of salinities in the model at various depths

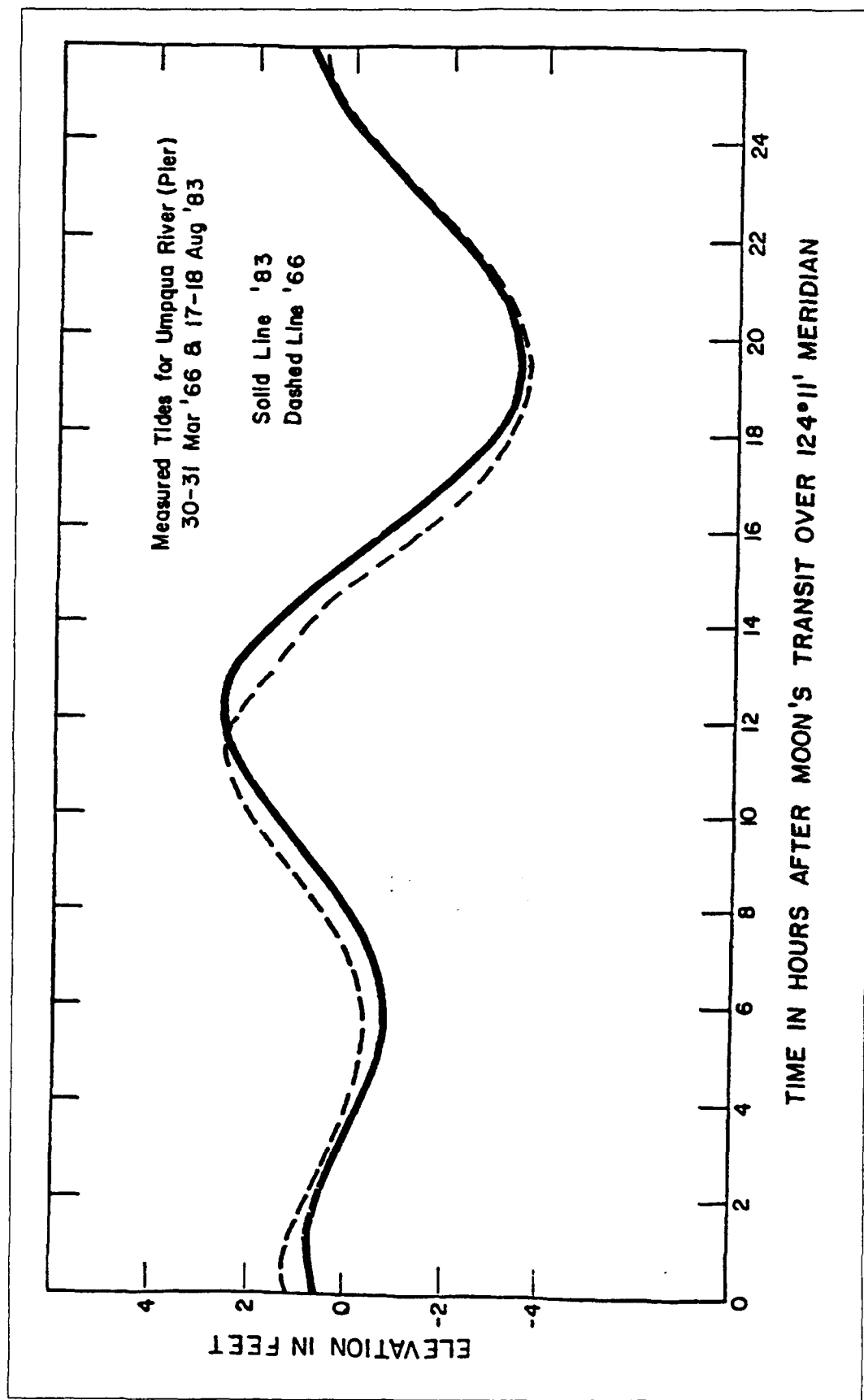Figure 31. Predicted tides at river entrance (1966 and 1983 prototype data)

Figure 32. Measured tides at river entrance (1966 and 1983 ptototype data)
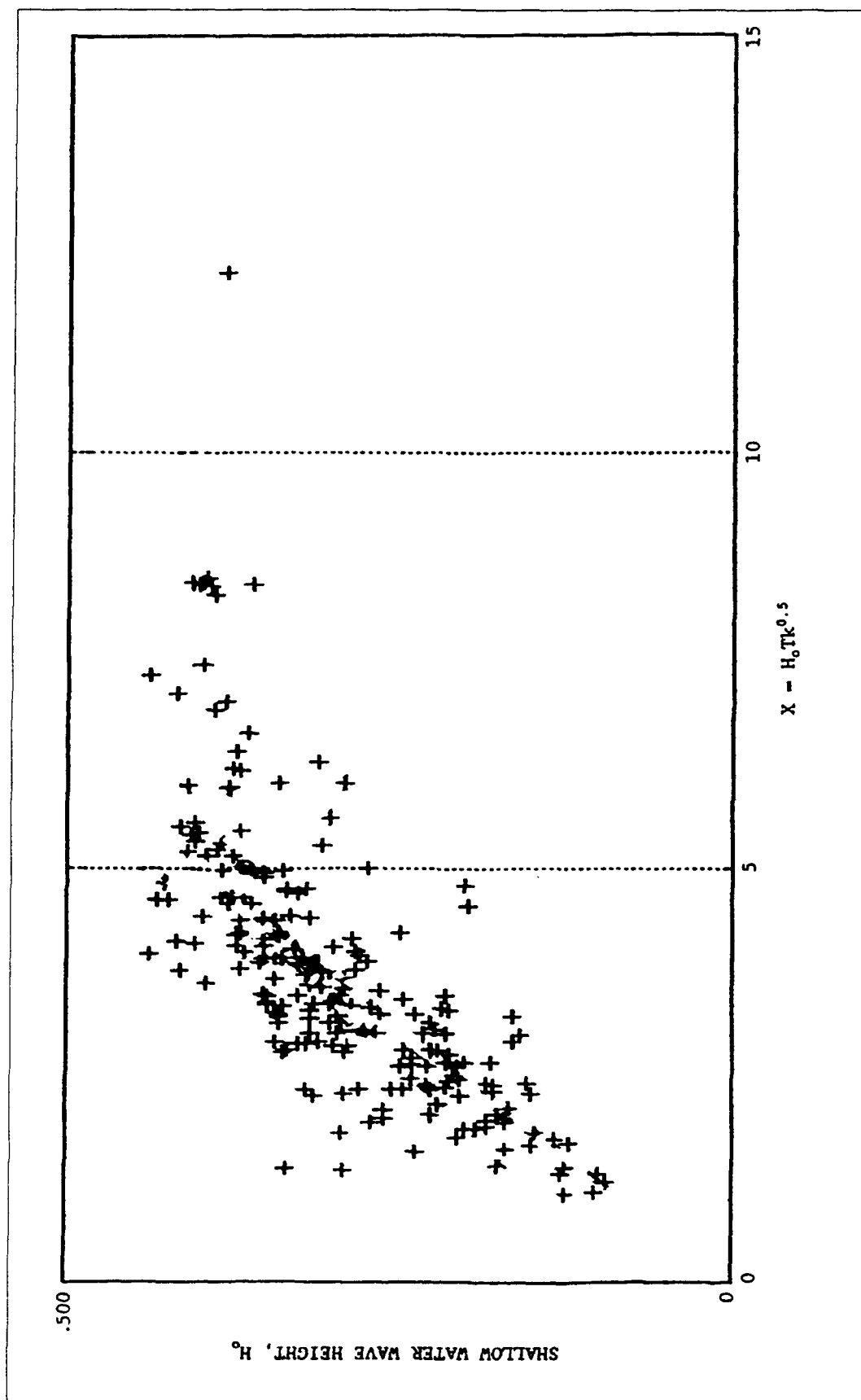
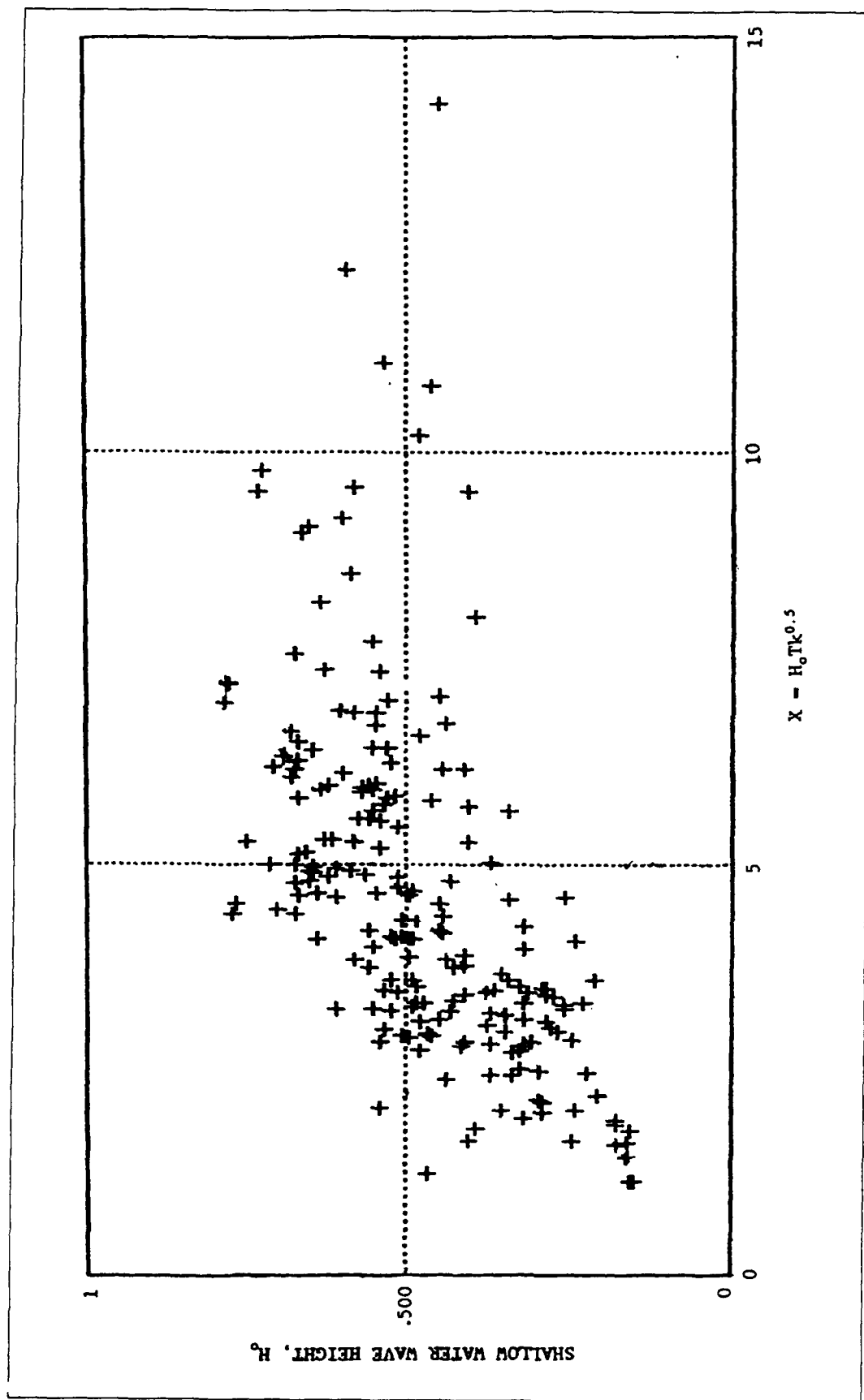Figure 33. Analysis of prototype daily average significant wave heights

Figure 34. Analysis of prototype daily maximum significant wave heights